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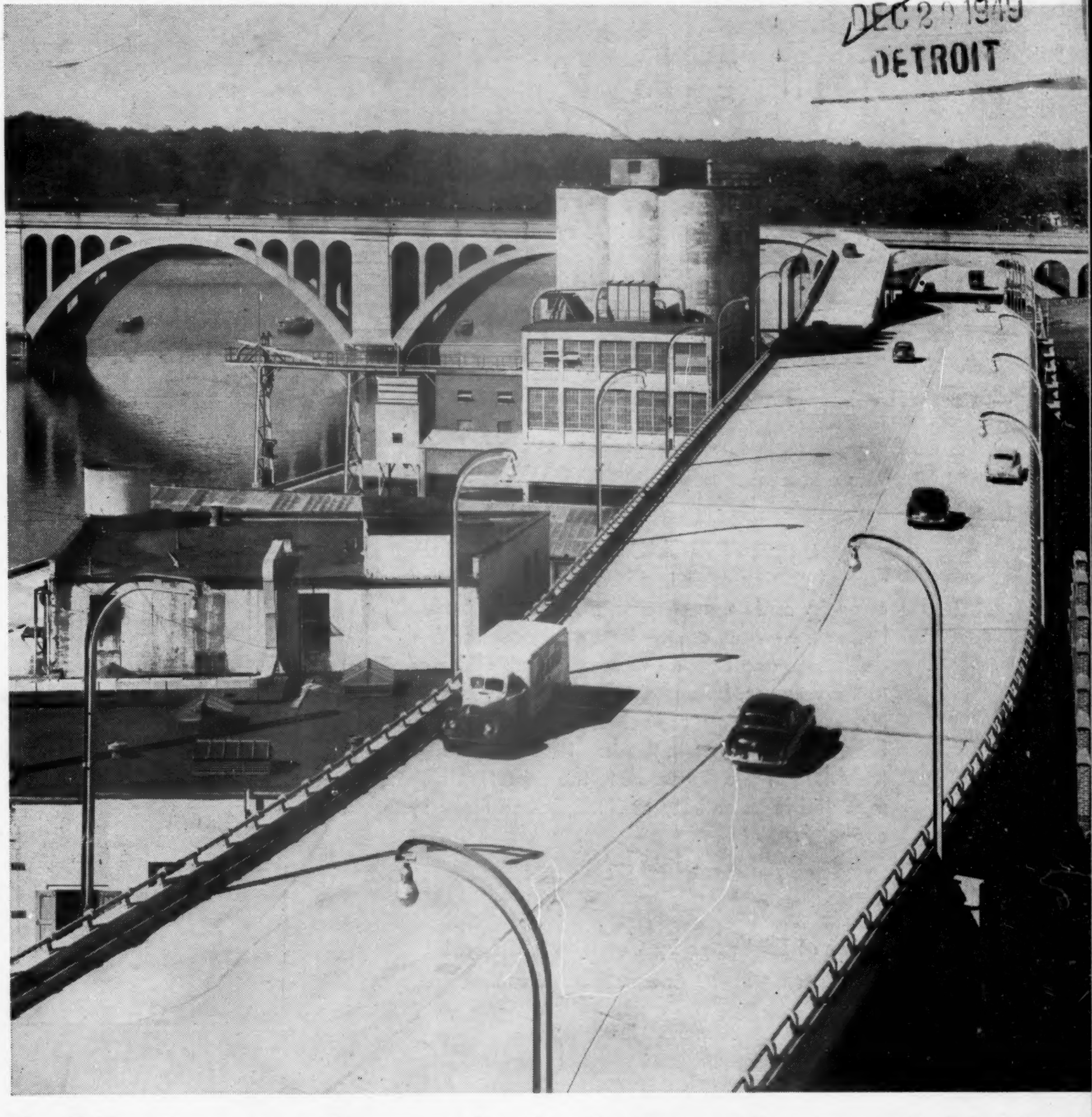

The efficiency of expressway traffic movement depends on the adequacy of the entrance and exit facilities

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BUREAU OF PUBLIC ROADS

U. S. DEPARTMENT OF COMMERCE

E. A. STROMBERG, Editor

IN THIS ISSUE

In this issue of PUBLIC ROADS appears the second portion of an important work on highway capacity and its practical applications. The first half of the report, containing an introduction and definitions and dealing with maximum observed traffic volumes, fundamentals of highway capacity, and roadway capacities for uninterrupted flow, was presented in the last issue of PUBLIC ROADS. At a future date the report will be reprinted in its entirety as a manual on highway capacity.

Highway Capacity:

Practical Applications of Research

BY THE COMMITTEE ON HIGHWAY CAPACITY
DEPARTMENT OF TRAFFIC AND OPERATIONS
HIGHWAY RESEARCH BOARD

Reported by
O. K. NORMANN, Chairman,
Chief, Section of Traffic Operations,
Highway Transport Research Branch,
Bureau of Public Roads
and W. P. WALKER, Secretary,
Highway Engineer,
Highway Transport Research Branch,
Bureau of Public Roads

Part V.—Signalized Intersections

INTRODUCTION

In part IV it was shown that the basic capacity of any highway facility is seldom achieved because of the effect of a large number of variable factors that tend to act as retardants to free vehicular movements. These variable factors were enumerated and measures of their effect have, in some cases, already been determined. One of the more important elements limiting the capacity of any facility, especially that of city streets, is the intersection at grade. Because of the many variations in the lay-out of intersections and the multiplicity of impediments which are usually inherent in city traffic, the capacity of intersections is here treated as a separate subject.

A city street loaded to its possible capacity during rush hour.

The Highway Capacity Committee found that very little material had been published on this subject, so it was necessary that some research be performed before satisfactory data could be assembled. In addition to the data collected by committee members, a large amount of material was furnished by most of the State highway departments, and by officials of many cities, as a result of solicitation by the Bureau of Public Roads. Some cities, including Chicago, Philadelphia, Milwaukee, and Washington, assigned several men to the task of obtaining information on a scale never before attempted. The analysis represents the conditions reported for hundreds of intersections and many months of tedious work by the Bureau of Public Roads. There will, of course, be additions and refinements to this report which may be issued as supplements as further analyses are made or as more data for specific conditions become available. There is little possibility, however, that additional data obtained within any reasonable length of time will materially change the present results.

There are certain factors that influence intersection capacities for which there are no available data. Particularly perplexing among these is the extent to which environmental and operating characteristics, which might vary widely between different localities, influence the number of vehicles that can pass through an intersection in a period of time. Local traffic regulations, the degree of enforcement, and the education and training of drivers are among the elements that comprise these environmental conditions which cannot be evaluated from the material at hand. Their average effect, however, is included.

The number of usable observations for some of the conditions covered is less than would be desirable, but the results have in all cases been based on far more information than has heretofore been available.

UNITS FOR EXPRESSING SIGNALIZED INTERSECTION CAPACITY

Current practice in expressing intersection capacity varies widely among individuals and this accounts in part for some of the controversies that have been known to develop in the past over the numbers of vehicles that can clear an intersection in an hour. It is important that the reader have an understanding of the units of measure used in this report, for otherwise the results are subject to misinterpretation.

Basically, an intersection consists of an intersection area and a number of legs or roadways on which vehicles approach and leave the intersection area. The traffic signals limit the number of approaches on which vehicles may move at one time, whereas the use of exit lanes is generally not restricted by the traffic signal.

At some locations, where all legs are not of the same width or where traffic backs up from an adjacent intersection, the capacity of the intersection may be dependent upon the capacity of the exit roadways. Generally, however, the capacity of the approaches controls the capacity of the intersection. It is also seldom that all approaches are burdened to their full capacities simultaneously. The maximum total volume from all approaches might conceivably occur when no one of the approaches was congested. It is appropriate, therefore, that intersection capacity be thought of in terms of the capacity of each of the approaches.

The number of vehicles that can enter an intersection is dependent upon a large number

A downtown street in a large city during an off-peak period. The traffic load is near the practical capacity of the intersection.

of factors, some of which are variable while others are fixed or semifixed. Mere numbers of vehicles are of little value unless they are accompanied by sufficient information to permit their evaluation in consideration of these factors. As a minimum requirement, a unit of time and a unit of roadway width are essential. These two units, together with the number of vehicles, can be incorporated in an expression of the rate of flow relative to time and street width.

That portion of time occupied by the red or STOP signal indication has no utility value insofar as the traffic which it holds at a standstill is concerned. This fixed portion of the time is lost to that traffic, there being a zero rate of flow. Hence, only the time during which the signal is green (the GO signal) is used in calculating rates of flow, and the hour of green is the unit in which it is expressed.

By applying the fraction of the total time that the signal is green for a particular movement to a known rate of flow in terms of vehicles per hour of green, the number of vehicles that can enter the intersection from that approach during 1 hour of total elapsed time can be readily calculated. In applying the capacity values given later in the report, the reader is cautioned against adding the amber time or any portion thereof to the green time in calculating the percentage of the total signal cycle during which traffic on one approach leg of the intersection is free to move. This would produce erroneous results because vehicles that entered during the amber period have been combined with those that entered during the green period to obtain the hourly rates shown in this report.

The basic unit of width used in expressing the capacity of roadways is the traffic lane. When the analysis of the intersection data was begun, it was also believed that the number of lanes would be a most important controlling factor and that streets of certain widths which would provide for an even number of 9- to 11-foot lanes would be more efficient than streets that were somewhat narrower or wider. It was believed, for example, that a street with a clear width between curbs of 20 feet for one lane in each direction or a clear width of 40 feet for two lanes in each direction would be much more efficient per foot of width than, for example, a street 27 feet wide for two-directional flow. Trial analysis of the data revealed, however, that intersection capacity varies in almost direct ratio with the width of the approach, measured from the curb line, and that the results for comparable traffic conditions were more consistent when based on the capacity per foot of width than when based on the capacity per traffic lane.

For example, a 64-foot street had the same capacity per foot of width if it was striped for either eight or six lanes, whereas the capacity per lane was greater for the six wider lanes than for the eight narrower lanes. With the wider lanes, the vehicles in the same lane travel closer together and fewer vehicles straddle the lane lines than they do when the lanes are narrow.

The basic unit "vehicles per 1 foot of width" is, however, somewhat unrealistic and involves very low traffic figures. The 10-foot width, which approximates the width of a lane as usually defined, has therefore been adopted as the most convenient unit of measure. It should be borne in mind, however, that the approach width used by traffic entering an intersection from one direction on a two-way street is normally one-half of the total street width unless there are safety zones or islands.

To summarize briefly the foregoing, **the capacity of signalized intersections is expressed as vehicles per 10 feet of width per hour of green.**

CAPACITY CLASSIFICATION

The problems encountered in moving traffic through intersections at grade vary in complexity from those usually found in rural areas, where the only interference to traffic is created by the cross movement of vehicles, to those found in urban areas, where there are pedestrians, cars entering and leaving the traffic stream on the approaches, and busses stopping to discharge or pick up passengers.

Basic Capacity

With a little imagination, it is possible to visualize a rural intersection with no interference to traffic except that created by the cross movement of vehicles. If this cross traffic is properly controlled by a traffic signal, the burden of the interference is transferred to that signal. The condition then existing is comparable to that at a signal installation in an isolated area with no cross movement, the only interference being that resulting from the periodic interruption to traffic when the signal turns red for traffic on the highway. This is the type of intersection where basic capacity can be realized and, as such, is worthy of brief consideration to discover how traffic performs at this ultimate rate of flow.

In part IV of this report it was stated that the basic capacity of a 12-foot traffic lane on a multilane highway is 2,000 passenger cars per hour. The inference might be that the basic capacity of a similar traffic lane at a

A streetcar loading island. Pedestrians crossing to the island are doing so in violation of the red signal indication.

signalized intersection is 2,000 vehicles per hour of green because under ideal conditions the signal would seem to be the only impediment, and that the detrimental effect of the red indication could be eliminated by converting the rate of flow to vehicles per hour of green time. This deduction is not entirely correct, however, because one of the conditions that must prevail for this high volume to be realized is the movement of all vehicles at a uniform speed of about 30 miles per hour. This uniform speed cannot be achieved if any single unit of the traffic stream is stopped for any reason.

With a low traffic volume, many vehicles will approach the intersection while the signal is green and proceed through at a reasonably high rate of speed. With heavier volumes of traffic, an increasingly large number of vehicles will be stopped by the red indication. Irrespective of the traffic volume, some vehicles will slow down when turning, or for pedestrians, thus affecting the whole traffic stream.

Parked vehicles deprive a street of a much greater portion of usable width than the space they actually occupy. This street is in an "intermediate area."

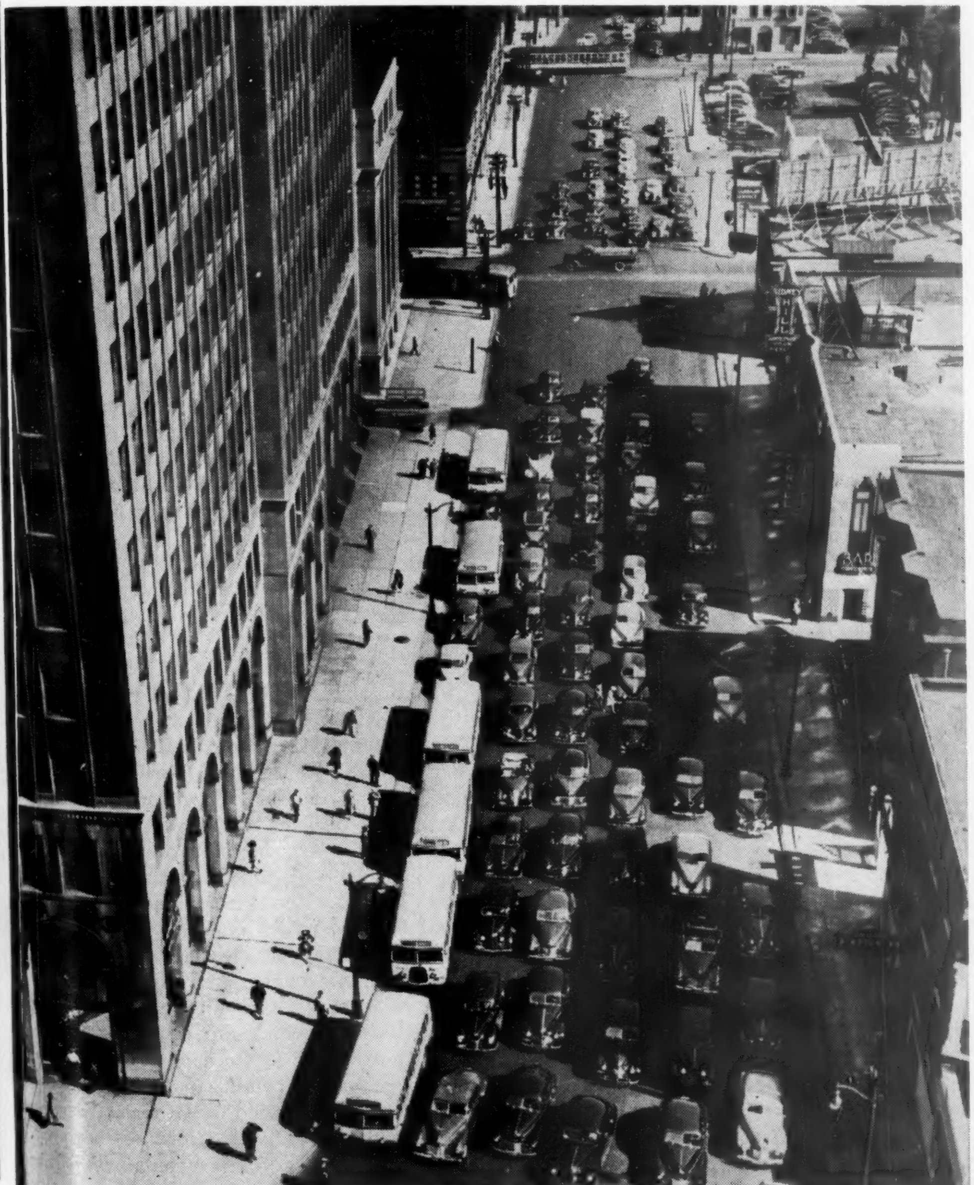
When the volume approaches the possible capacity of the intersection so that every green interval is fully utilized, there will always be some vehicles waiting to enter the intersection as the light turns green. Under this condition, a relatively high percentage of the cars approaching the intersection will be required to stop even on a street with a synchronized or progressive signal system.

With most vehicles coming to a stop, the average speed at which they can cross an intersection is 10 to 15 miles per hour. With traffic moving at a speed of 12 miles per hour, the highest rate at which a single line of vehicles can enter an intersection is 1,500 passenger cars per hour. As a confirmation of this figure, experience has shown that the minimum spacing between passenger cars as they start from a standing position one behind the other in a 12-foot lane averages about 2.4 seconds. The time intervals for the first two vehicles in line are usually considerably greater than 2.4 seconds, but between succeeding vehicles the interval decreases progressively until it reaches an average minimum of 2.1 seconds between the fifth and sixth cars in line.

Within the range of vehicles normally accommodated by one signal cycle, 2.4 seconds is a representative average headway. This headway corresponds to an hourly rate of 1,500 vehicles. **It follows, then, that the basic capacity of an isolated signalized intersection is about 1,500 passenger cars per hour of green per 12-foot lane, or 1,250 vehicles per 10 feet of width per hour of green.** This figure corresponds to the highest rates at which vehicles have been observed to pass a point on a roadway removed from the influence of intersections after being stopped. It also corresponds to the rates attained for each movement through a signalized intersection where there are separate signal indications and traffic lanes for each traffic movement, and few pedestrians, so that the permitted movements do not interfere with one another.

Possible Capacity

Basic capacity can be attained only under the ideal conditions previously described. Inherent in any traffic stream, however, are several causes for minor delays, such as stalled motors and inept drivers, which cannot be eliminated by any practical means.



High traffic density on a one-way street in a downtown area. Where busses continuously load and unload, a negligible number of passenger cars utilize the first lane.

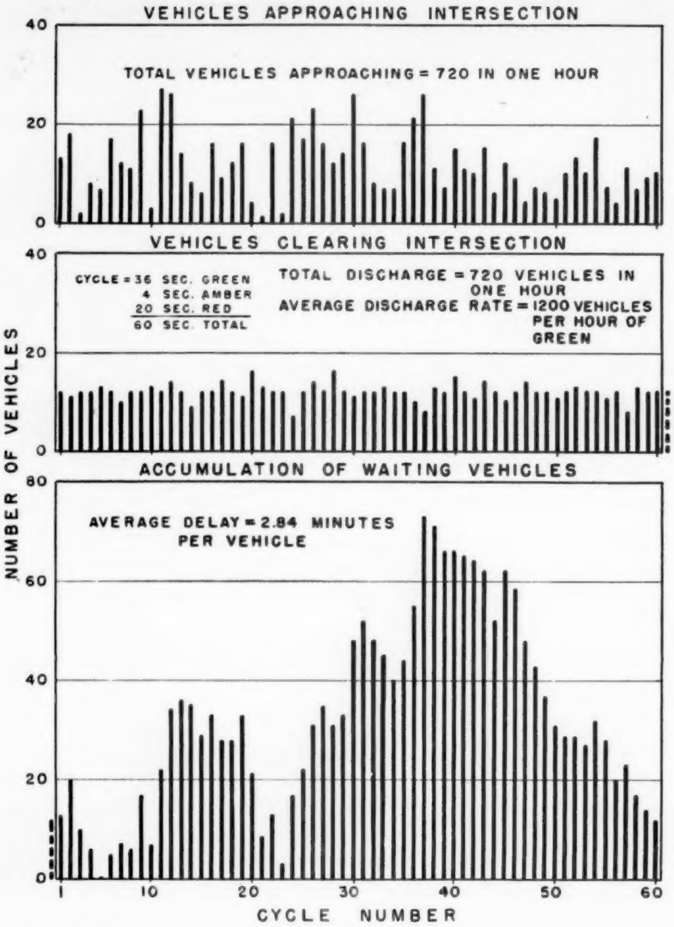


Figure 21.—Operation of traffic at a rural intersection loaded to its possible capacity.

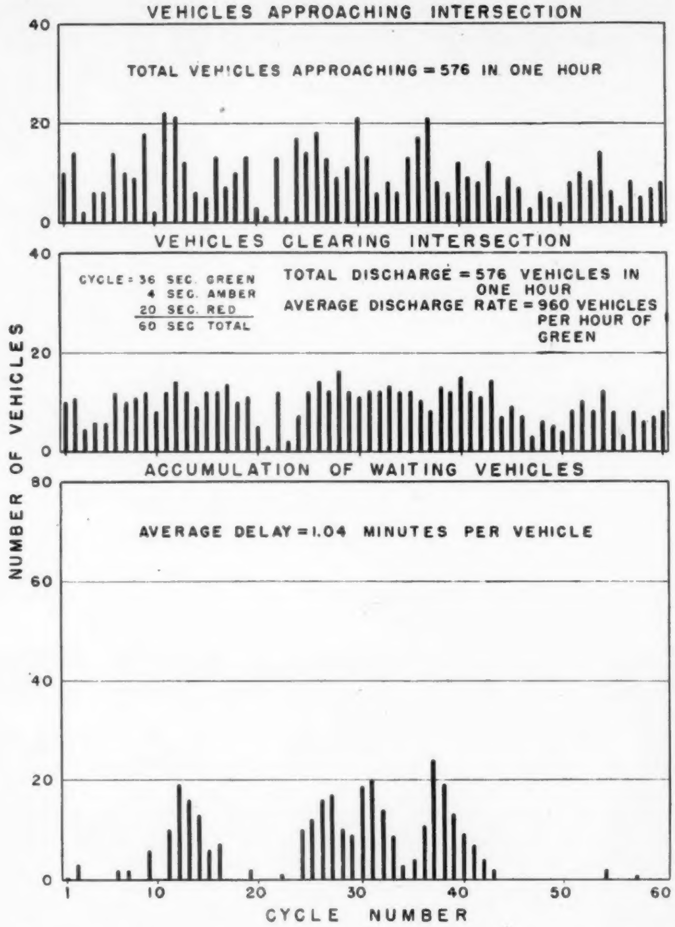


Figure 22.—Operation of traffic at a rural intersection loaded to its practical capacity.

These conditions are included with the ideal in determining basic intersection capacity.

Basic capacity, however, does not include any reduction in capacity caused by the following, which are the principal factors that tend to reduce traffic flow at intersections:

1. Parked vehicles and vehicles being parked or leaving parking areas.
2. Turning movements.
3. Commercial vehicles, including streetcars.
4. Pedestrian interferences.
5. Inclement weather conditions.

When the basic capacity of an intersection is reduced by the effect which each of these factors that are present has on the traffic flow, the possible capacity of the intersection is obtained. **The possible capacity of an intersection approach, therefore, is the maximum number of vehicles that actually can be accommodated under the prevailing conditions with a continual backlog of waiting vehicles.**

Practical Capacity

When the traffic volume on any intersection approach is sufficient to tax the approach to its possible capacity during every signal interval for an hour—that is, when there is always a backlog of waiting vehicles—there will be times when the queue of waiting traffic will become extremely long, resulting in lengthy and intolerable delays for a large number of drivers.

This is illustrated graphically by figure 21, which shows the length of back-up, in terms of numbers of vehicles, that occurred at an intersection approach loaded to its possible capacity. Traffic approaching the intersection from one direction was of sufficient volume to utilize the full green period during each signal cycle for a full hour. Some drivers in this particular example were required to wait through six signal cycles before reaching and proceeding through the intersection. Such lengthy delays are intolerable and the traffic volume at which they occur can certainly not be considered practical. Yet, because of the normal short-time variation in traffic flow on approach highways or streets, this is the typical condition which must and does occur at an intersection when each green period of a fixed-time signal is fully utilized for an entire hour.

The practical capacity of an intersection approach is the maximum volume that can enter the intersection from that approach during 1 hour with most of the drivers being able to clear the intersection without waiting for more than one complete signal cycle. With the normal short-time variation in flow, practical intersection capacities have been found to be approximately 80 percent of the possible capacities. In other words, at most intersections few vehicles will be required to wait for more than one signal cycle if the hourly approach volume is 80 percent of the possible capacity. The comparatively little delay when the approach volume is 80 percent of the possible capacity is illustrated by figure 22 in which the approach volume is 80 percent of the approach volume shown in figure 21. It should be noted that all of the green periods were not fully utilized under the conditions represented by figure 22.

Many attempts to determine intersection capacities have been based on rates attained during green periods that were fully utilized. Other attempts have been made by expanding the average rate attained during the five or ten green periods in 1 hour when the largest number of vehicles passed through the intersection. The first method completely ignores the normal short-time fluctuation in traffic flow, whereas the second method fails to recognize the effect of inept drivers and a variety of other factors that are ever present in any traffic stream and cannot be eliminated. As a result, theoretical intersection capacities are calculated which are much higher than those possible of attainment even under the best of prevailing conditions.

The results obtained by either of these two methods are highly factitious and approach the basic capacity of intersections. Before these extremely high volumes could be attained in practice it would be necessary to eliminate those factors that cause any reduction in flow

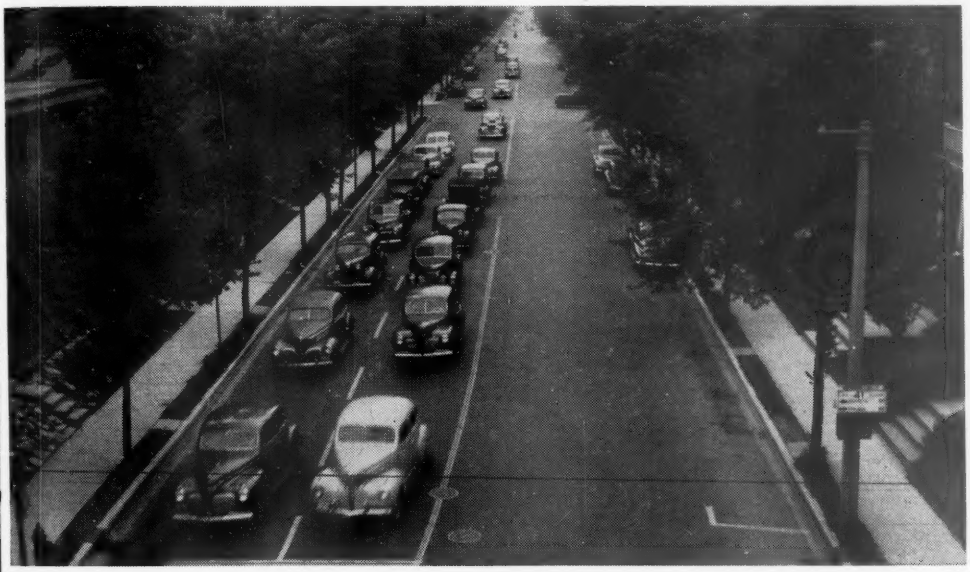
below the rates attained during short peak periods. Among these factors are the stalling of motors, stopping to pick up passengers, and traveling at spacings greater than those used by the small group of reckless drivers.

CLASSIFICATION OF INTERSECTION TYPES

When the engineers throughout the country were asked to furnish data on observed intersection volumes that could be used to determine intersection capacities, they were requested to submit the highest traffic count made at each intersection during a period of 1 hour while most of the green intervals for at least one approach were fully utilized and a high percentage of the drivers had to wait for at least one full signal cycle before entering the intersection. It is evident, therefore, that the data upon which the following results are based represent volumes exceeding the practical capacities. It is not likely, however, that all the data represent possible capacities with all green intervals for an hour fully utilized. This condition rarely occurs even at the most congested intersections. The average condition represented is, therefore, somewhere between the possible and practical capacities. Were this not true for data obtained at the most congested intersections throughout the country, there would be no traffic problem at rural or urban intersections.

One hour is generally the shortest period of time that conforms to current traffic-counting practice. The hourly rates shown as a result of these investigations, therefore, include the short-time variations that normally occur in the flow of traffic. When applying the results to intersections where there are exceptionally heavy surges of short duration or where the flow actually exists for less than an hour, such as at an entrance or exit at a factory, the capacity rates must be calculated for a like fractional part of an hour.

A street intersection in a downtown area during a peak period.



This intersection, in an intermediate area, is loaded beyond its practical capacity. Parking is not permitted on the approach in this block.

It is extremely difficult to isolate each of the variable factors that reduce traffic flow at signalized intersections and to determine the extent to which each influences the capacity of intersection approaches of different widths. Efforts to segregate their effect one from the other are often frustrated by the fact that they are in many cases very closely interrelated as, for example, the number of right turns and the number of pedestrians crossing the intersecting street. Even where the factors affecting capacity can be segregated and evaluated, the different conditions under which they may later be applied are so varied in character that the resulting estimates of capacity should be considered as approximations only.

For example, in the discussion of turning movements which appears later, each 1 percent of left-turning vehicles is said to reduce the capacity of an intersection approach by 1 percent. Although this factor is correct for the average intersection under average condi-

An intersection in an outlying area, with added turning lanes for left-turn movements and separated turning lanes for right-turn movements.

An intersection in a downtown area of a small city during an off-peak period when parking was permitted.

tions, its application will only produce reasonably accurate results for all conditions.

Precise results cannot be expected in a broad application to intersections of varied width and having different types of traffic, for many reasons. Among these are:

1. If two or more successive vehicles desire to turn left, the effect per vehicle on the capacity of the street is not as great as if the vehicles turned at more widely spaced intervals. The larger the number of left-turning vehicles, the lesser the effect per vehicle.
2. The effect on capacity of left-turning vehicles is related to the number of oncoming vehicles going straight through and turning left.
3. Each left-turning vehicle crosses the path of pedestrians moving with the green light, and therefore the effect of a left turn is to some extent dependent on the number of pedestrians.
4. A vehicle waiting to make a left turn causes a greater relative reduction in capacity on a narrow street than on a wider street or on one having a wide center-dividing island.

It is obvious that the correction factors needed to meet the possible combinations of the five elements enumerated under the discussion of possible capacity would total a very sizable number. Rather than attempting to apply each of these varied correction factors, it is much more feasible that intersections be grouped into certain classes or categories for which the combined effect of several elements which usually occur in about the same combination can be easily determined or applied. The capacity of any intersection that does not fall directly within a particular class can be determined with a reasonable degree of accuracy by interpolation.

The variables by which intersections have been classified are as follows:

1. *Street width:* Various ranges of street width within which the capacity per 10 feet of width shows little variation when other conditions are comparable.
2. *Type of area:* Downtown, intermediate, or outlying.
3. *Parking regulations:* Parking prohibited or permitted.
4. *Streetcars:* With or without streetcars.

These variables are sufficiently descriptive to classify most intersections. It would have been desirable to have had further breakdowns for items such as the percentage of commercial vehicles, the presence of bus-loading zones, the percentage of vehicles involved in turning movements, the extent to which the parking spaces were utilized, and the number of pedestrians. The four variables listed above, however, are sufficiently descriptive to classify most intersections so that the actual capacity for any condition can be estimated with a reasonable degree of accuracy. Within the downtown area, for example, most available parking spaces will be utilized and there will generally be a large number of pedestrians during the peak hour. If this is not true at the particular intersection under consideration, a capacity either in the upper or lower part of the range for the downtown intersections would be more applicable than the average value.

Table 17.—*Average of maximum observed capacities at urban signalized intersections free from streetcars and curb parking on the approach streets: For approaches loaded beyond their practical capacities*

	Downtown area	Intermediate area	Outlying area
VEHICLES PER HOUR OF GREEN PER 10 FEET OF APPROACH WIDTH [1]			
Total street width, curb to curb:			
31–46 feet	717	888	703
47–64 feet	740	595	638
65 feet and over	704	538	---
All widths	730	695	673
AVERAGE PREVAILING CONDITIONS			
Pedestrian crossings	2, 200	880	200
Percentage of vehicles:			
Turning right	12	11	11
Turning left	13	10	13
Percentage of commercial vehicles	11	10	9

[1] Approach width assumed to be one-half of total street width because these results include two-way streets only.

An intersection on a high-type facility. The traffic volume is near the practical capacity of the intersection, although between intersections the highway is adequate for a much greater volume.

HIGHEST OBSERVED VOLUMES FOR AVERAGE CONDITIONS

Capacities where parking is prohibited

Table 17 shows the average traffic volumes recorded at a large number of intersections in cities distributed throughout the country, during periods in which the volumes approached the capacity of the intersections. These results are for four-way intersections controlled by fixed-time signals and free from streetcars and parked vehicles on the approach streets. The range of volumes averaged in table 17 is shown in figure 23.

The difference between the figures shown in table 17 and the basic intersection capacity of 1,250 vehicles per hour of green per 10 feet of width can be attributed to the combined effect of pedestrians, turning movements, commercial vehicles, short-time fluctuations in flow, and other causes as they were present at the average intersection of each type. In downtown areas the reduction below basic capacity from these causes averages about 40 percent, while in intermediate and outlying areas it is slightly larger, being about 43 percent.

In the downtown areas, there is little difference in the volume of traffic handled per unit of width by the narrow and the wider streets, whereas in the intermediate and outlying areas the narrower streets handled somewhat more traffic per unit of width than the wider streets. A number of explanations can be offered for this characteristic difference between street widths in the various areas, but such explanations are of little practical value unless the effect of each cause can be established and applied when estimating capacity volumes for specific intersections. The averages of the more important conditions affecting intersection capacities that prevailed at locations in each area where parking is prohibited are shown at the bottom of table 17.

Capacities where parking is permitted

Table 18 shows the average traffic volumes recorded at intersections where parking was permitted at the curb on the approach streets. Except for this feature, these intersections are comparable in all other respects to the intersections for which data are shown in table 17. It is emphasized that the rates are for the entire approach width, including the portion of the street occupied by the parked vehicles. This basis for showing the results was used so that the total effect of the parked vehicles could be obtained without introducing errors by deducting an assumed street width for the space they occupy. The range of observed volumes averaged in table 18 is shown in figure 23.

Comparison of the data shown in tables 17 and 18 reveals that capacity volumes on downtown streets where parking is permitted are about 55 percent, on the average, of those observed where parking is prohibited. In intermediate areas the effect of curb parking is somewhat less, causing an average reduction of about 35 percent. In outlying areas, where but few vehicles park, street capacity is generally not impaired by vehicles parking or unparking. The occasional driver who does desire to park is usually able to pull directly into a parking space instead of backing in as is required in areas where most parking spaces are occupied.

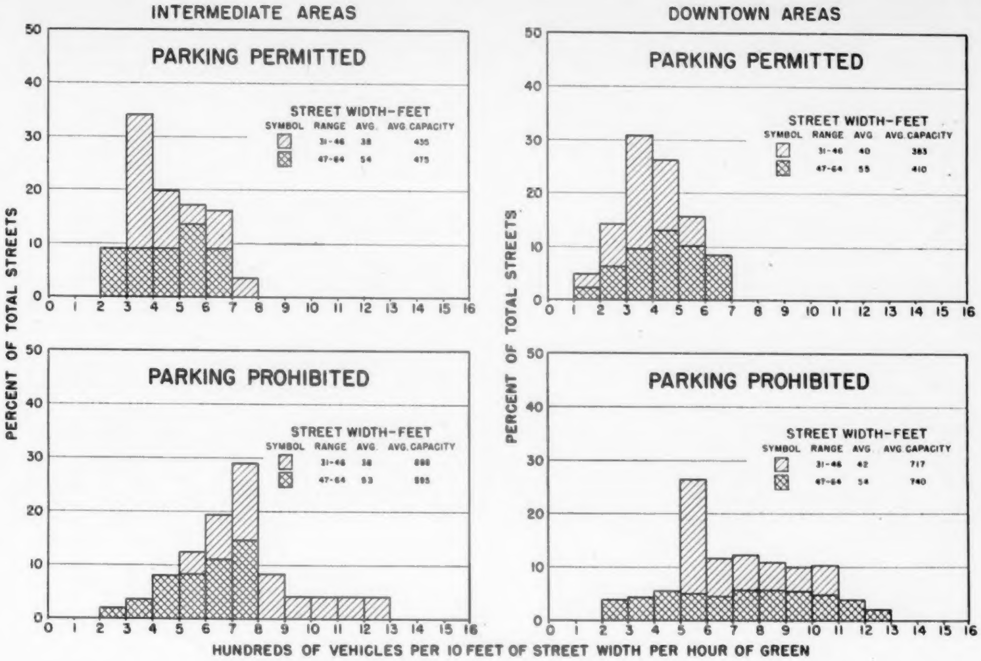


Figure 23.—Frequency distribution of intersection capacities.

Table 18.—*Average of maximum observed capacities at urban signalized intersections free from streetcars but with curb parking permitted on the approach streets: For approaches loaded beyond their practical capacities*

	Downtown area	Intermediate area	Outlying area
VEHICLES PER HOUR OF GREEN PER 10 FEET OF APPROACH WIDTH [1]			
Total street width, curb to curb:			
31-46 feet	383	435	732
47-64 feet	410	475	618
65 feet and over	400	470	---
All widths	398	453	692
AVERAGE PREVAILING CONDITIONS			
Pedestrian crossings	2,700	500	200
Percentage of vehicles:			
Turning right	17	6	14
Turning left	9	8	11
Percentage of commercial vehicles	11	10	12

[1] Approach width assumed to be one-half of total street width because these results include two-way streets only.

Heavy bus traffic in a downtown area. At this intersection busses load and unload at both the curb and the outer separator.

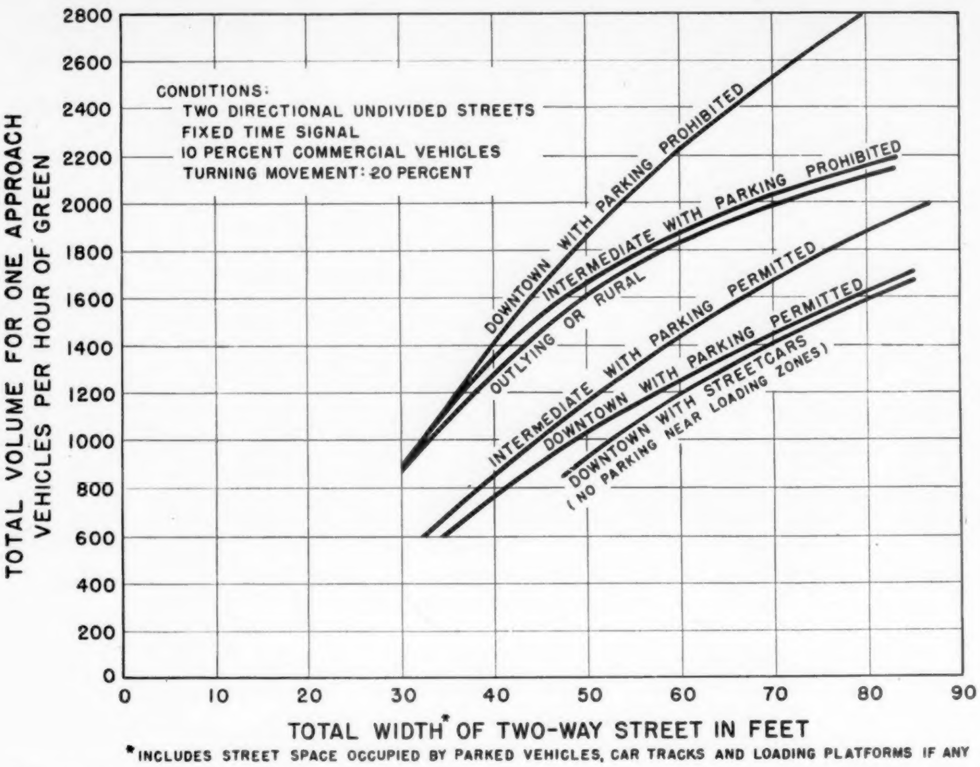


Figure 24.—Average reported intersection capacities for two-way streets by type of area and parking regulation.

Curb parking has less effect on street capacities in intermediate areas away from business centers than in downtown areas because fewer cars are found at the curb even though it may be perfectly legal to park. Also the turnover is not as great, resulting in less interference to the moving traffic.

It is especially significant that whatever the width of a downtown street, intersection capacities where curb parking is permitted are from 43 to 47 percent below the capacities for similar streets where curb parking is prohibited. The difference for the two conditions is not due entirely to the street space occupied by the parked vehicles. Also included is the combined effect of the many other factors that tend to restrict traffic flow which are present where parking is permitted, such as the interference to through traffic caused by drivers maneuvering to enter or leave parking spaces and the tendency of drivers to shy away from

Table 19.—*Average reported capacities of downtown streets at intersections where there are two car tracks and a loading island with no parking adjacent to the island*

Street width		Vehicles per hour of green per 10 feet of width	
Curb to curb	Curb to island	Based on total width of approach	Based on width between curb and platform
Feet	*Feet*		
58	14	408	857
64	17	403	760
70	20	385	685
90	30	383	575

parked vehicles to avoid the possibility of hitting a car starting to pull out, or a pedestrian coming from between the parked vehicles.

That a line of vehicles parked adjacent to the curb reduces the effective street width more than 7 or 8 feet becomes evident when street widths necessary to accommodate the same traffic volumes at locations where parking is permitted and where parking is prohibited are compared. Figure 24 shows, for example, that the average 40-foot street with no parking in a downtown area can accommodate the same volume of traffic as the average 68-foot street on which parking is permitted. This is a reduction of 14 feet in the effective width of the 68-foot street for each line of parked vehicles.

Figure 24 shows graphically the relation between the average reported intersection approach capacities and the total street width for the three types of areas. The conditions for which the curves apply are listed on the chart. It will be noted that these conditions are representative of those occurring at the average intersection, a separate curve being shown for streets on which streetcars operate.

The conditions listed on figure 24 are not all of those prevailing that affected the maximum observed volumes during periods that traffic flow exceeded the practical capacities. In the average downtown area, for example, traffic regulations are more rigidly enforced, available parking spaces are more fully utilized, and there is more stopping to unload and pick up passengers, than in the intermediate areas. The differences between these conditions in the two types of areas account in part at least for the higher average capacities on streets of the same widths in the intermediate areas than in the downtown areas when parking is permitted, whereas the reverse is true for the streets when parking is prohibited. In other words, street capacities are increased less by prohibiting parking in areas where a low demand for parking exists and parking restrictions are not enforced, than in areas where the demand is great and parking restrictions are enforced.

Capacity of streets having streetcars

The presence of streetcars can affect the capacity of a street in a number of ways, depending on local practices and regulations as well as on structural features such as loading islands. Headway between streetcars and the question of whether or not free-wheeled vehicles are permitted use of the car-track lanes are matters that must be considered in addition to all the elements that pertain to the capacity of streets where there are no streetcars.

The following characteristics are peculiar to most downtown intersections where congestion is a problem and where there are streetcar tracks:

1. Loading islands or safety zones are provided on the intersection approach.
2. Free-wheeled vehicles are not permitted to use the car-track lane adjacent to the loading island.
3. Curb parking is not permitted for a distance equal to and usually in excess of the length of the loading island.

Table 19 shows average reported capacities for downtown intersection approaches where the foregoing conditions prevail. These figures are for straight-through tracks only. If the intersection is blocked by streetcars turning, the time during which free-wheeled vehicles are prevented from moving for this reason should be deducted from the available green signal time. The range of volumes averaged in table 19 is shown in figure 25.

It will be noted that the capacity per 10 feet of width decreases rapidly with an increase in the distance between the curb and

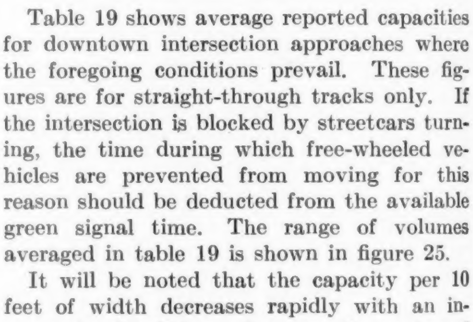


Figure 25.—Frequency distribution of observed intersection capacities on downtown streets with streetcars.

the loading platform. This suggests that pedestrians going to and from the platform may interrupt the flow of traffic to a greater extent when the street width between the curb and platform is wide than when it is narrow.

EFFECT OF VARIABLE CONDITIONS ON CAPACITY

Among the variable conditions that affect capacity of intersections are the proportion of commercial vehicles in the total traffic volume, the existence of bus stops, the extent of turning movements, the type of signal system, and the use of one-way operation. These conditions, and their effects on the capacity of intersections of the common type, are considered in the following paragraphs. Thereafter, the capacity of intersections on high-type facilities is discussed.

Common-Type Intersections

Commercial vehicles

The presence of commercial vehicles tends to reduce intersection capacities in terms of the total number of vehicles because their acceleration rates are lower and they occupy more road space than passenger cars. On the average, one commercial vehicle, not including those that stop to pick up or discharge passengers or goods, is equivalent in an intersection capacity sense to two passenger cars. This is true only when they are not involved in turning movements or where the turning movements can be made without more than normal interference with other vehicles.

Bus stops

The effect of busses stopping to load and unload passengers on the capacity of a specific intersection depends to a large extent on the many different conditions present at the intersection. Analyses of the available data show the following results for the average conditions prevailing at signalized intersections where the total street width is under 60 feet:

1. Streets in the downtown and intermediate areas where parking is permitted, except at a near-side bus stop, have a 12-percent higher capacity than the streets where busses do not load and unload and parking is not restricted near the intersections.

2. In the downtown and intermediate areas where parking is prohibited, the streets with bus stops on the near side of the intersection have a 15-percent lower capacity than the streets where there are no bus stops. The corresponding figures for far-side bus stops are 8 percent in the downtown areas and 20 percent in the intermediate areas.

There is evidence, therefore, that elimination of parking near the intersection to provide a bus stop helps more to increase the capacity than the stopping of the busses decreases the capacity. Also, on streets where parking is prohibited, the results indicate that to achieve high capacities bus stops in the downtown area should be on the far side of the intersections wherever possible while stops outside of the central business district should be on the near side of the intersections. It is recognized that considerations other than street capacity

A Y intersection on an expressway.

often govern in the selection of bus stop location. Insufficient data are available to draw any conclusions for streets over 60 feet wide or for the relative merits of near- and far-side stops on streets where parking is permitted.

The above figures apply only to conditions found at the average intersection on streets in the two types of areas. There are many specific conditions that are far from average. Probably the one condition which is apt to be farthest from average at a specific location is the number of busses. A few studies conducted on the same streets during periods of normal operation and again during periods when the bus operators were on strike show that one bus has the same effect on intersection capacity as three to five passenger cars. These results apply only to locations where parking was prohibited for the entire distance between intersections and relatively short stops were made by the busses to pick up and discharge passengers.

Observations also indicate that on wide multilane streets where there are a large number of busses, or the busses stop for long periods to take on and discharge passengers, the busses in each direction reduce the effective street width for passenger cars by at least 12 feet.

Turning movements

The extent to which turning movements reduce the capacity of intersections is dependent upon such conditions as the intersection lay-out or treatment, pedestrian movement, the volume of oncoming traffic, and, of course, upon the number of vehicles turning right or left. In most instances, the detrimental effect of turning movements in rural areas can be greatly reduced through proper channelization and widening of the pavement. Under adverse conditions, the effect of turning movements may be sufficiently great to reduce the practical capacity of an intersection on a two-lane road or street by as much as 50 percent. On a multilane facility under similar conditions, the left lane may have its capacity reduced by 50 percent or more as a result of heavy turning movements.

The only general criterion obtained by analyzing the available intersection data was

An intersection with a separated right-turning lane.

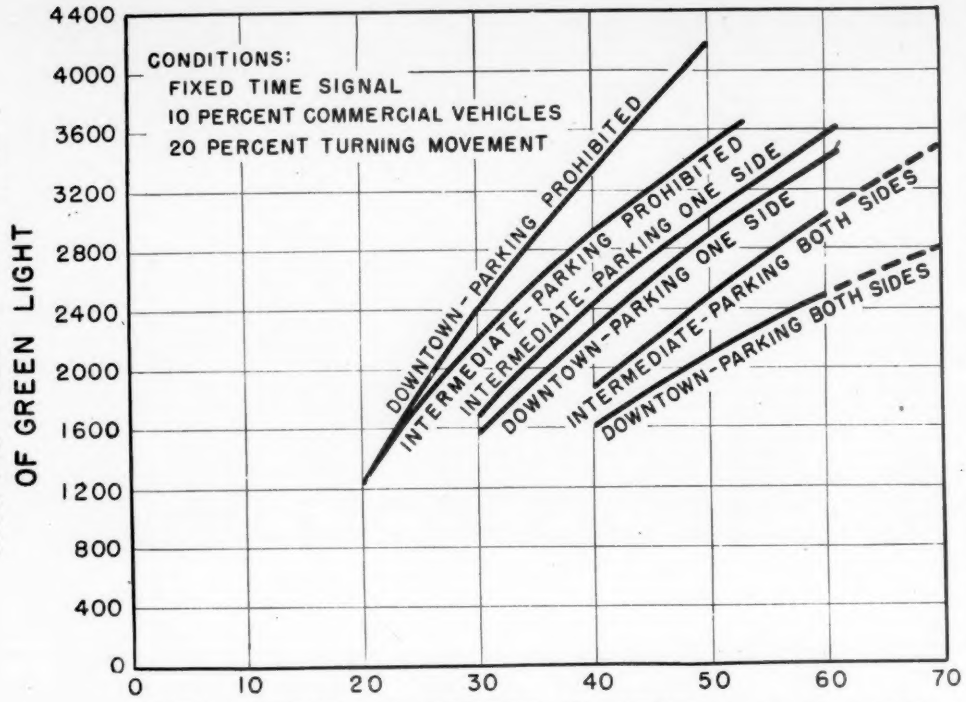


Figure 26.—Intersection capacities of one-way streets by type of area and parking regulation

that each 1 percent of the total traffic turning right reduced the capacity flow ½ percent, and that each 1 percent of the total traffic turning left reduced the capacity flow 1 percent. These percentages apply only where there is no separate signal indication for the turning movements and where all turns comprise less than 30 percent of the total traffic. Above 30 percent, no additional decrease in volume was apparent.

Where there are separate signal indications for turning movements, careful investigations are usually required and it is difficult to formulate specific rules for these situations. As a general rule, the total street width is proportioned between the various movements in the ratio of the number of lanes utilized by traffic in the performance of these movements. The capacity for each separate movement is then computed, using as a basis the length of the interval and width of street allotted to the particular movement. The capacity of the intersection approach is a summation of the computed capacities for all movements emanating from that approach. Because the object of separate signal indications is the elimination of conflicts between various movements, the rate of flow for turns will generally be the same as that for straight-through movements.

Signal system

The type of signal system on a street makes little difference in the possible capacity of the facility. Although a perfectly planned progressive signal system may function well with low or even moderately heavy traffic volumes, some traffic will be stopped by the red light at each signal when the traffic volume becomes sufficient to tax the capacities of the intersections. Vehicles following on the next band will be slowed by these waiting vehicles and the system is immediately thrown off balance. The higher speed for which the system was planned cannot be regained until the traffic density becomes less than that which occurs when the street is carrying near capacity loads. The benefits accruing to vehicle operators as a result of a progressive signal system are reflected in a saving in time to the traffic using the facility, but this time saving declines rapidly when the practical capacity is exceeded.

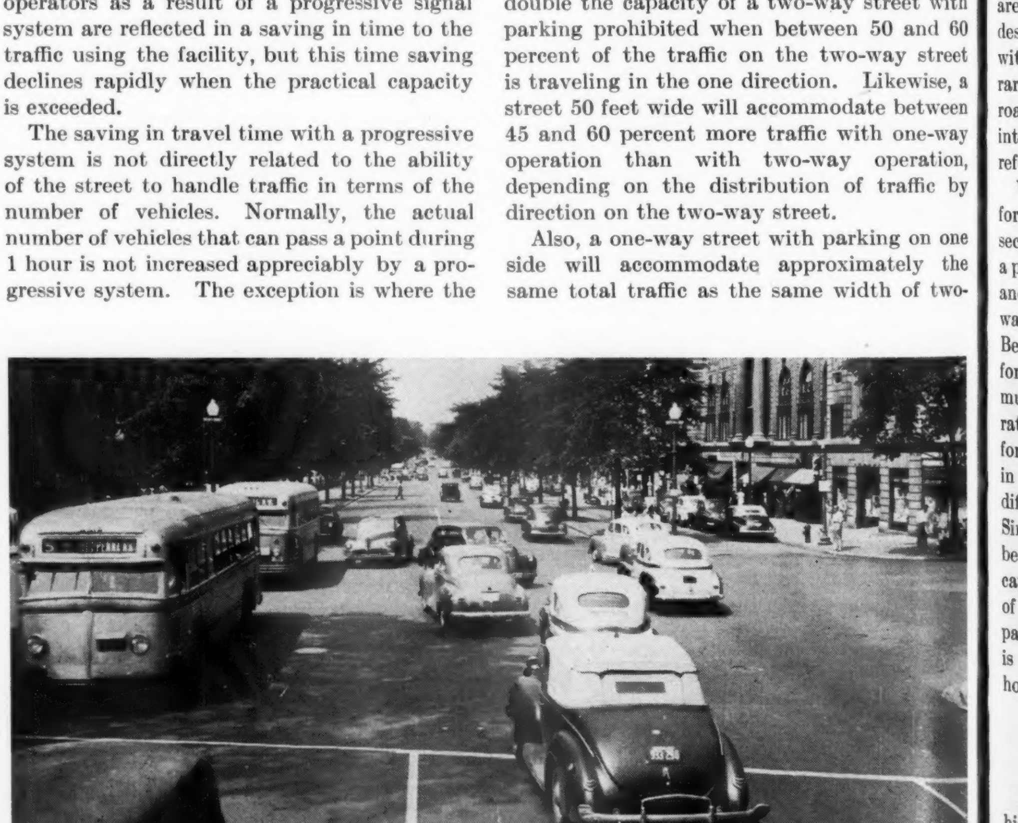
The saving in travel time with a progressive system is not directly related to the ability of the street to handle traffic in terms of the number of vehicles. Normally, the actual number of vehicles that can pass a point during 1 hour is not increased appreciably by a progressive system. The exception is where the distance between traffic signals is so short that the road space between them will not accommodate the number of vehicles passing through one intersection and headed for a second intersection during one complete signal cycle. This being true, there is a possibility that sounder principles may be developed for adjusting a progressive system for *peak loads* as a result of these investigations.

One-way operation

Figure 26 shows intersection capacities for one-way streets by type of area and parking regulation when 10 percent of the traffic is commercial vehicles and 20 percent of the total traffic is involved in turning movements. The advantage of one-way streets over two-way streets from the capacity viewpoint is illustrated for certain conditions by figures 27 and 28. The advantage of one-way streets over two-way streets will vary with the distribution of traffic by direction on the two-way streets, the relative number of turning movements involved, and the width of the streets.

Figure 27 shows the advantage of one-way streets over two-way streets in the downtown area where two-way streets normally carry between 50 and 60 percent of the traffic in one direction during the peak periods. Except for the streets over 60 feet wide with parking on both sides and a 50–50 distribution of traffic by direction on the two-way streets, the capacity of a one-way street is higher than the capacity of a two-way street. For example, a one-way street 20 feet wide with parking prohibited has approximately double the capacity of a two-way street with parking prohibited when between 50 and 60 percent of the traffic on the two-way street is traveling in the one direction. Likewise, a street 50 feet wide will accommodate between 45 and 60 percent more traffic with one-way operation than with two-way operation, depending on the distribution of traffic by direction on the two-way street.

Also, a one-way street with parking on one side will accommodate approximately the same total traffic as the same width of two-



The effect of left-turning vehicles on intersection capacities is dependent upon a large number of variable conditions.

way street with no parking. The two-way streets are considered to be at their capacity volumes when the traffic in the direction of the heavier flow equals the capacity of the approaches used by traffic traveling in that direction.

Figure 28 shows the comparative capacities of one-way and two-way streets in intermediate areas where the volume in one direction includes up to 67 percent of the total flow on two-way streets. For this condition there is also a greater advantage in making the narrow streets one-way than in making the wider streets one-way. It may be noted that the total capacity of a two-way street with a 67–33 distribution of traffic by directions is the same when parking is prohibited as when parking is permitted on the side carrying the lower traffic flow. Also, a one-way street 50 feet wide with parking on both sides has the same capacity as a 50-foot two-way street either with no parking or with parking on one side.

Intersections on High-Type Facilities

At highway intersections where pedestrian interference for the most part is eliminated, where separate lanes for each traffic movement are provided so that the permitted movements do not interfere with one another, where commercial vehicles are not present, and where the geometric features of the highway are of a high standard, a basic capacity of about 1,500 vehicles per hour of green per 12-foot lane may be approached. The type of facility to which this is applicable would be a divided highway, in a rural or an urban area, with added turning lanes of adequate design provided for both right and left turns, with shoulder space for disabled or temporarily stopped vehicles, and with no other roadside interference on the approaches of the intersection—that is, a facility normally referred to as an expressway.

Where such conditions exist, the method for capacity analysis given for street intersections is not directly applicable. Instead, a procedure of starting with the basic capacity and reducing this value to fit prevailing roadway and traffic conditions is suggested. Because express-type facilities are intended for rapid vehicular movement with a minimum of operational delay, practical capacity rather than possible capacity should be used for design. Under the conditions outlined in the preceding paragraph there is little difference between basic and possible capacity. Since practical capacity at intersections has been found to be about 80 percent of possible capacity, it follows that the practical capacity of a 12-foot traffic lane is 1,500×0.80=1,200 passenger vehicles per hour of green. This is equivalent to 1,000 passenger vehicles per hour of green per 10 feet of width.

ADJUSTMENTS FOR SPECIFIC CONDITIONS

The ability of intersections to accommodate high hourly traffic volumes has been grossly overrated in the past. The application of attainable capacities, which in some cases are so low as to be astonishing, will be an important step in alleviating congestion on existing facilities and in estimating the needed improvement for future requirements.

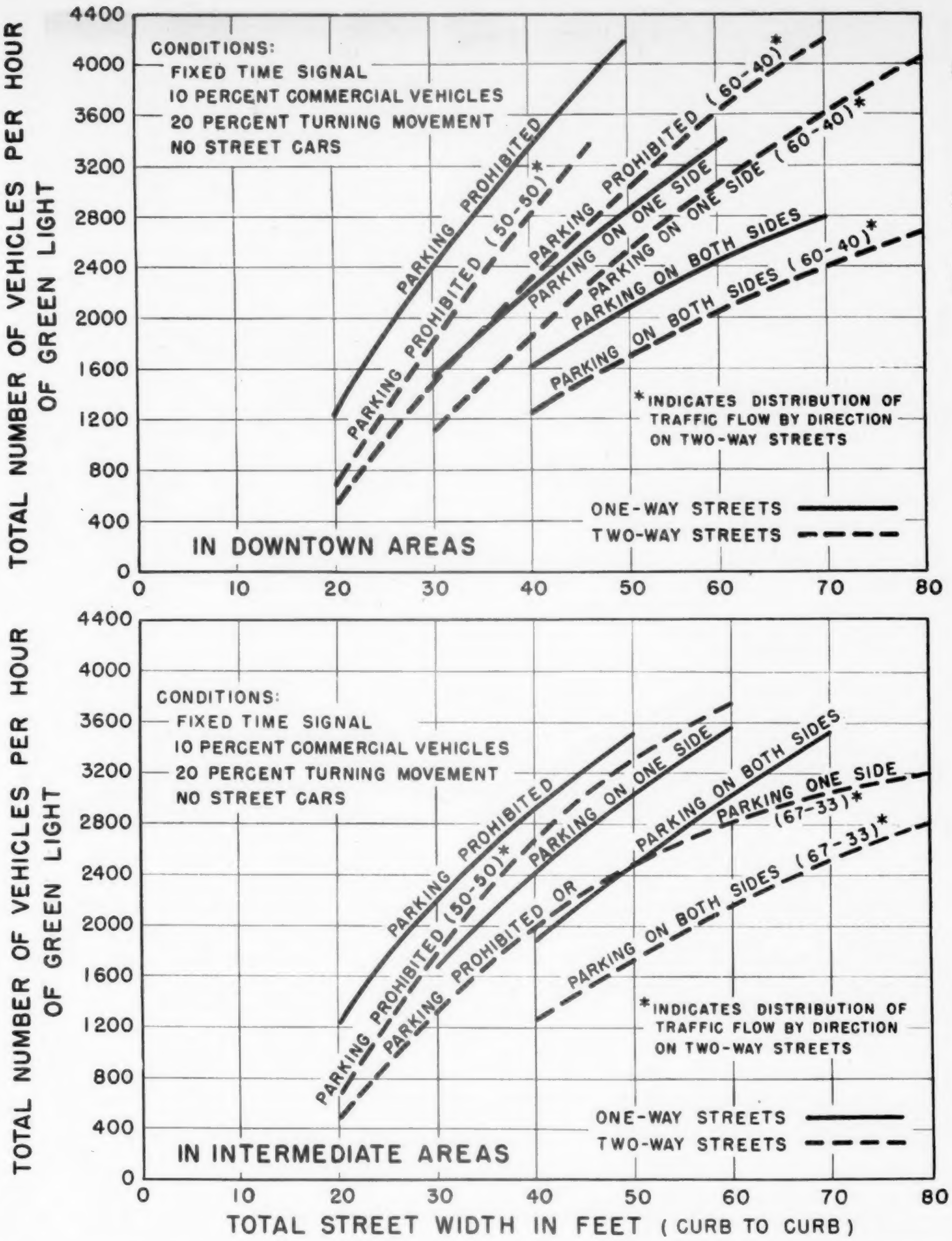


Figure 27 (**above**) *and figure 28* (**below**).—*Comparison of average street intersection capacities with one-way and two-way operation, in downtown and in intermediate areas.*

As previously stated, the intersection capacities that have been presented are averages for maximum recorded traffic volumes under the conditions that prevailed at the average intersection of each general class. They are peak volumes during periods that the approaching traffic exceeded the practical intersection capacities. Since there is a wide range in the maximum volumes for each group into which the intersections have been classified, it is essential, when estimating the capacity of any specific intersection, that the average figures as presented be adjusted to provide for the difference between average conditions and the existing or future conditions at the specific location. Figures 23 and 25, which show the distribution of observed rates at congested intersections, illustrate the need for making such adjustments.

Rates exceeding those for the average intersection are applicable only when conditions which restrict the flow of traffic are not present in the same degree as at the average intersection of the same class. Likewise, when the restricting conditions are more pronounced than at the average intersection, and this will be the case about 50 percent of the time, rates below average should be applied.

The highest rates recorded are about 700 vehicles per 10 feet of street width per hour of green or go time in areas where parking is permitted between intersections, and about 1,250 vehicles per 10 feet of street width per hour of green in areas where parking is prohibited. These rates cannot occur where the various movements interfere with one another, where pedestrians are present, or where

***Flexible lane usage:** Normally this is a two-way street, but during the morning rush there are four lanes provided for south-bound traffic and one lane, restricted to local and bus traffic, for north-bound vehicles. The operation is reversed for the afternoon peak load. Widely spaced vehicles suggest moderately high speeds, which is indicative of heavy volume with little congestion. Note that the single parked vehicle has deprived traffic of the use of a full lane and, further, that traffic passing the standing vehicle allows a wide margin of clearance.*

trucks or busses are present. Even under the most ideal conditions for high capacities where these rates did occur, operating conditions could not be considered satisfactory.

Intersections on existing or contemplated facilities should not be expected to accommodate the peak capacities that have been recorded at the few locations where conditions were most favorable for high rates of flow. Such an expectation would probably result in operating conditions that would be less favorable than those now present in our most congested areas.

If it were possible to list all the factors that tend to reduce intersection capacities, and the quantitative effect of each, the most appropriate procedure to follow in estimating the capacity of a specific intersection would be to start with the capacity for ideal conditions and deduct a certain amount for each of the prevailing conditions that are not ideal. This procedure is not possible of application, however, because a quantitative measure has been obtained for only the more important factors that affect intersection capacity.

A number of adjustments are necessary when applying the information for average intersection conditions, as shown by figure 24, to a specific location where conditions are not average. The most important of these adjustments are described here.

I.—Two-way streets with no added turning lanes and no separate signal period for turning movements

1. Possible and practical capacities:

A. *Possible capacity.*—On an average, possible capacities are about 10 percent higher than the average rates represented in figure 24. Volumes 10 percent higher will pass through the intersection, but only with a continuous back-log of vehicles and extremely long delays to a high percentage of the drivers.

B. *Practical capacity.*—On an average, practical capacities are about 10 percent lower than the average rates represented in figure 24. Volumes 10 percent lower will pass through the intersection with few drivers having to wait longer than for the first green period.

2. Commercial vehicles:

Subtract 1 percent for each 1 percent by which commercial vehicles exceed 10 percent of the total number of vehicles, or add 1 percent for each 1 percent that commercial vehicles are less than 10 percent of the total.

3. Turning movements:

A. *Right turns.*—Subtract ½ percent for each 1 percent by which traffic turning right exceeds 10 percent of the total traffic, or add ½ percent for each 1 percent that traffic turning right is less than 10 percent of the total (maximum reduction for right turns not to exceed 10 percent).

B. *Left turns.*—Subtract 1 percent for each 1 percent by which traffic turning left exceeds 10 percent of the total traffic, or add 1 percent for each 1 percent that traffic turning left is less than 10 percent of the total (maximum deduction for left turns not to exceed 20 percent).

Note.—Maximum deduction for right and left turns combined should not exceed 20 percent.

4. Bus stops and elimination of parking near intersection:

A. *On streets where parking is prohibited.*—

(a) No bus stop: Add 5 percent.

(b) Bus stop on near side: Subtract 10 percent.

(c) Bus stop on far side: Subtract 3 percent in downtown areas and 15 percent in intermediate areas.

(d) Where the number of busses is so great that at least one is always loading or unloading, subtract 12 feet from each approach width for either near- or far-side stops when applying the curves in figure 24; then add the number of busses and make the adjustments for items 1, 2, and 3, above, but do not include the busses as commercial vehicles in item 2.

B. *On streets with bus stops and where parking is permitted except at the bus stop.*—

(a) With bus stop on near side: Add ¼ percent for each 1 percent of right and left turns, but maximum increase not to exceed 6 percent.

(b) With bus stop on far side: Make no correction.

(c) Where the number of busses is so great that at least one is always loading or unloading: Subtract 6 feet from the approach width for either near- or far-side stops when applying the curves in figure 24; then add the number of busses and make the adjustments for items 1, 2, and 3, above, but do not include the busses as commercial vehicles.

C. *On streets where parking is permitted and there are no bus stops.*[11]—Deduct ¼ percent for each 1 percent that right and left turns combined are of the total traffic, but maximum deduction not to exceed 6 percent. Then, if parking is prohibited more than 20 feet in advance of the cross walk, add

$$P\left(\frac{D-20}{5G}\right) \text{ percent, where}$$

P=total percentage of right and left turns, but not to exceed 30.

D=distance in feet that parking is prohibited in advance of cross walk, but not to exceed $5G$ plus 20.

G=seconds of green indication per signal cycle.

[11] The discussions concerning the needed length of no-parking zones in advance of cross walks, the capacity of added turning lanes (item II), the effect of separate signal indications (item III), and the procedure for determining capacities of intersections on one-way streets (item IV) and on high-type facilities (item V), represent a rationalization based on such facts and data as are available concerning these important subjects, for which the reported material is insufficient for statistical analysis.

D. *On streets where parking is eliminated for a limited distance on both sides of the intersection.*—Where parking is eliminated in advance of the intersection for a distance in feet equal to or greater than $5G$ and (1) parking is also eliminated beyond the intersection for a distance equal to or greater than $5G$, or (2) the street beyond the intersection widens at least one lane: Use the upper curve (parking prohibited) in figure 24.

II.—Two-way streets with added turning lanes but no separate signal indication for the turning movements [12]

1. Use the width of the through lanes as one-half of the street width when applying the curves in figure 24.

2. Then add 5 percent for an added right-turn lane, 10 percent for an added left-turn lane, or 15 percent when both right- and left-turn lanes are added to the normal width.

3. Then:

(a) *For a right-turn lane.*—Add the number of vehicles turning right but not to exceed either $600 \times \frac{G}{C}$ vehicles per hour or $\frac{D-20}{25} \times N$ vehicles per hour, where

G=green interval in seconds.
C=total signal cycle in seconds.
D=length of added turning lane in feet.
N=number of signal cycles per hour.

(b) *For a left-turn lane.*—Add the number of vehicles turning left but not to exceed the capacity of the left-turn lane. The capacity of the left-turn lane per hour of green in terms of passenger cars may be estimated as the difference between 1,200 vehicles and the total opposing traffic volume per hour of green in terms of passenger cars, but not less than two vehicles per signal cycle.

4. Then adjust as in I–1 and I–2.

III.—Two-way streets with turning lanes and separate signal indication [12]

Streets with turning lanes and separate signal indication (also applies where left-turn lanes for opposite directions of travel are within the normal street width and both straddle the centerline); pedestrians controlled:

A. *With right-turn lane.*—

(1) Use the width of the through lanes as one-half of the street width when applying figure 24 and increase the rate of flow by 5 percent.

(2) Then add the number of vehicles turning right, but not more than 800 vehicles per 10 feet of width of turning lane per hour of separate green indication; adjust for possible or practical capacity as in item I–1 and for commercial vehicles as in item I–2.

B. *With left-turn lane.*—

(1) Use the width of the through lanes as one-half of the street width when applying figure 24 and increase the rate of flow by 10 percent.

(2) Then add the number of vehicles turning left, but not more than 800 vehicles per 10 feet of width of turning lane per hour of separate green indication; adjust for possible or practical capacity as in item I–1 and for commercial vehicles as in item I–2.

Note: Where conditions are such that through vehicles also use turning lanes, it is more appropriate to apply figure 24 to (1) entire approach width for period of left-turn indication and (2) width of through lanes for balance of through period. Add results and make normal adjustments except that for left turns 10 percent is always added.

IV.—One-way streets [13]

Figure 26 shows hourly intersection capacities for urban one-way streets by type of area and parking regulation under average conditions. The most important adjustments for conditions that are not average are:

1. Possible and practical capacities:
Same as for item I–1.

2. Commercial vehicles:
Same as for item I–2.

3. Turning movements:
Subtract ½ percent for each 1 percent by which the combined traffic turning right and left exceeds 20 percent of the total traffic, or add ½ percent for each 1 percent that it is below 20 percent (maximum deduction for turns not to exceed 20 percent).

4. Bus stops and elimination of parking near the intersection:
Same as for item I–4.

5. Added turning lanes:
Use the normal street width when applying the curves in figure 26. Before applying items 1 and 2 above:

(a) Add 5 percent for an added right- or left-turn lane or 10 percent if both a right- and left-turn lane have been added.

(b) For a right- (or left-) turn lane add the number of vehicles turning right (or left) but not to exceed either $600 \times \frac{G}{C}$ vehicles per hour or $\frac{D-20}{25} \times N$ vehicles per hour.

V.—High-type facilities [13]

Where conditions exist similar to those previously described for intersections on high-type facilities, the following procedure for estimating practical or design capacities should be used:

1. Through movement:
Use 1,000 vehicles per hour of green per 10 feet of lane width. Deduct 1 percent for each 1 percent that commercial vehicles are of the total through movement during the peak hour.

2. Turning movements on added lanes:
To determine the total capacity of an intersection approach, add the number of vehicles turning right and left during the hour to the capacity of the through flow; however, each turning volume should not exceed the capacity of a turning lane as determined below. Also, the added turning lane should be of sufficient length to accommodate at least twice the average number of turning vehicles that would accumulate during the red interval.

A. *Right turns on same signal indication with through movement.*—

(1) Where there is no adjacent frontage road and no pedestrian interference, use capacity of turning lane as 1,000 vehicles per hour of green per 10 feet of width. Deduct 1 percent for each 1 percent that commercial vehicles are of the right-turning traffic during the peak hour.

(2) Where right turns are in conflict with frontage-road traffic, use capacity of right-turning lane as the difference between 1,200 vehicles and the total conflicting traffic volume (expressed in terms of passenger vehicles) on the adjacent frontage road per hour of green; adjust this difference by deducting 1 percent for each 1 percent that commercial vehicles are of the right-turning traffic during the peak hour. The volume thus determined must not be greater than that estimated under item (1) above, but not less than two vehicles per signal cycle.

(3) Where right turns are in conflict with pedestrian movements on the cross street, reduce flow estimated under item (1) above as follows:

Downtown area ... 20 percent.
Intermediate area ... 10 percent.
Outlying area ... no reduction.

(4) Where right turns are in conflict with both frontage-road traffic and pedestrians, use the lower of the two values estimated under items (2) and (3) above.

B. *Left turns on same signal indication with through movement.*—Use capacity of left-turning lane as the difference between 1,200 vehicles and the volume of the opposing through traffic movement (expressed in terms of passenger vehicles) per hour of green; adjust this difference by deducting 1 percent for each 1 percent that commercial vehicles are of the left-turning traffic during the peak hour; minimum capacity not less than two vehicles per signal cycle.

C. *Added turning lanes on separate signal indication (pedestrians controlled).*—For either right or left turns, use capacity of 1,000 vehicles per hour of separate green indication per 10 feet of lane width; deduct 1 percent for each 1 percent that commercial vehicles are of the particular turning traffic during the peak hour.

3. Bus stops on added turning lane:

A. *Far-side bus stop.*—No apparent effect on capacity of the intersection approach.

B. *Near-side bus stop (no separate signal indication for right turns).*—Bus stops on the right-turning lane tend to reduce the capacity of the through movement by requiring some right-turning movements to be made around the bus directly from through traffic lanes. Capacity in such case may be adjusted as follows:

(1) Where the number of busses stopping during the peak hour is so great as to nullify the use of the added lane for right turns, deduct 1½ percent from the

[12] See footnote 11.

[13] See footnote 11.

through flow for each 1 percent that right-turning traffic is of the total traffic; then add the number of busses and the number of vehicles turning right and left to obtain the total practical capacity of the intersection approach.

(2) Where approximately one bus stops in the added lane per cycle, deduct ½ percent from the through flow for each 1 percent that the right-turning traffic is of the total traffic; then add the number of busses and right- and left-turning vehicles to obtain total practical capacity.

(3) Where busses stop less frequently than one about every fourth or fifth cycle, the effect of busses may be neglected.

APPLICATION OF INTERSECTION CAPACITY INFORMATION

When adjusting the volumes shown in figure 24 for conditions that are not average, each adjustment must be made as a separate step, using the result of the previous step for each consecutive adjustment. To accomplish this simply, when a number of adjustments are necessary, each adjustment can be calculated and added to or subracted from 1.00, and a total factor then obtained from these individual factors by multiplying them together. The examples that follow illustrate correct applications of the data.

Example 1

Problem

What are the possible and practical capacities of one approach to an intersection on a two-way street, 45 feet wide from curb to curb, in a downtown area where parking is prohibited, 20 percent of the traffic turns right, 15 percent turns left, 5 percent of the total traffic is commercial during peak hours, there are bus stops on the near side of the intersection, and the traffic light has a GO or green period of 35 seconds out of the 60-second cycle?

Solution

From figure 24, the reported capacity of one approach on the average street 45 feet wide from curb to curb in a downtown area with parking prohibited is 1,660 vehicles per hour of green.

The following adjustments are required because conditions are not average:

Cause	*Effect*	*Factor*
Right turns	(10−20) ½ = − 5%	0.95
Left turns	(10−15) = − 5%	0.95
Commercial vehicles	(10− 5) = + 5%	1.05
Near-side bus stop	−10%	0.90
Total factor = 0.95×0.95×1.05×0.90 =		0.85

Possible capacity $= 1.10 \times 0.85 \times \frac{35}{60} \times 1{,}660 =$ 905 vehicles per hour in the direction of the heavier flow.

Practical capacity $= 0.90 \times 0.85 \times \frac{35}{60} \times 1{,}660$ $= 741$ vehicles per hour in the direction of the heavier flow.

Example 2

Problem

What is the practical capacity of two intersecting streets, if the north-south street is 40 feet wide and the east-west street is 54 feet wide? Parking is permitted on the east-west street and banned on the north-south street. The intersection is in a downtown area where average conditions prevail and the signal cycle is 27 seconds green and 3 seconds amber on each leg, with a total of 60 seconds.

Solution

From figure 24, the practical capacity of the north-south street is $0.90 \times 1{,}410 = 1{,}269$ vehicles per hour of green. The traffic signal reduces this volume to $1{,}269 \times \frac{27}{60} = 571$ vehicles per hour in one direction.

From figure 24, the practical capacity of the east-west street is $0.90 \times 1{,}110 = 999$ vehicles per hour of green. This is reduced by the traffic signal to $999 \times \frac{27}{60} = 450$ vehicles per hour in one direction.

Example 3

Problem

What should be the distribution of signal time at the intersection described in example 2 if the peak hourly volume in one direction on the north-south street is 600 vehicles per hour and that on the east-west street is 400 vehicles per hour?

Solution

The minimum time in minutes per hour for each operation, assuming a 60-second cycle, is as follows:

		Minutes
Green, north-south street	$\frac{600}{1{,}269} \times 60 =$	28.4
Green, east-west street	$\frac{400}{999} \times 60 =$	24.0
Amber: 6 seconds per cycle	=	6.0
Total		58.4

The green period for the north-south street should be:

$$\frac{28.4}{(28.4+24.0)} \times (60-6.0) = 29.3 \text{ seconds.}$$

The green period for the east-west street should be:

$$\frac{24.0}{(28.4+24.0)} \times (60-6.0) = 24.7 \text{ seconds.}$$

Example 4

Problem

An intersection as described in example 3 is severely congested by volumes exceeding those used in that example. To relieve this situation it is proposed that parking be banned on both streets, and that the north-south street be widened. After these improvements, traffic volumes of 750 vehicles per hour in one direction on the east-west street and 900 vehicles per hour in one direction on the north-south street are anticipated. What should be the width of the north-south street and the signal timing for the intersection, using an 80-second cycle with 3 seconds of amber?

Solution

Total amber time per hour will be $6 \times \frac{3{,}600}{80} =$ 270 seconds = 4.5 minutes.

A 54-foot street with no parking has a capacity of 2,000 vehicles per hour of green,

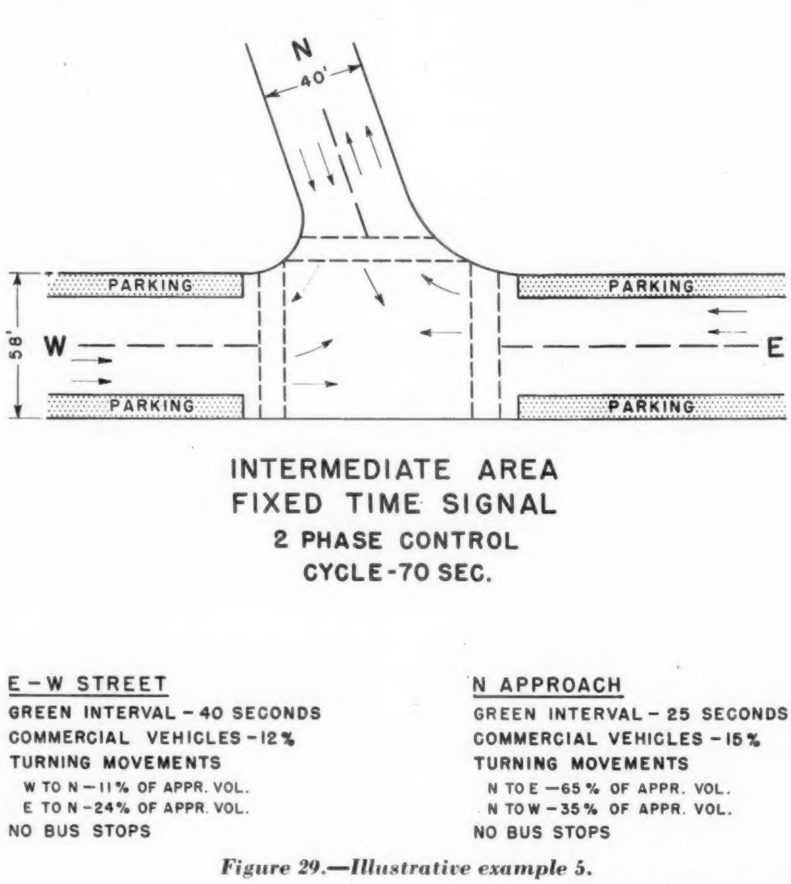


Figure 29.—Illustrative example 5.

from figure 24. Thus the east-west traffic will require $\frac{750}{0.9\times 2{,}000}\times 60=25.0$ minutes. There remain 30.5 minutes or $\frac{30.5}{60}$ hours of green on the north-south street, permitting a practical capacity of $\frac{60}{30.5}\times 900=1{,}770$ vehicles per hour of green. This practical capacity corresponds to a reported capacity for average conditions of $\frac{1{,}770}{0.9}=1{,}967$ vehicles per hour of green. A street 52 feet wide will accommodate this volume, as shown in figure 24.

The signal timing should be:

	Seconds
Green, north-south street $\frac{30.5}{(30.5+25.0)}\times(80-6.0)=$	40.7
Green, east-west street $\frac{25.0}{(30.5+25.0)}\times(80-60)=$	33.3
Amber ---------------------- $2\times 3.0=$	6.0
Total ----------------------------	80.0

Example 5

Problem

What are the possible and practical capacities of each approach of the T intersection shown in figure 29? If the peak-hour traffic (on the basis of two-way flow) is 9 percent of the average daily traffic, and 60 percent of the peak-hour traffic is in one direction, what will be the average daily traffic on the east-west street in the block west of this intersection if it operates at possible capacity?

Solution

West Approach

From figure 24, the reported capacity of one approach on the average 58-foot street in an intermediate area with parking is 1,380 vehicles per hour of green.

Adjustments:

Cause	*Effect*	*Factor*
Commercial vehicles	$(10-12)=-2\%$	0.98
Right turns	$(10-0)\frac{1}{2}=+5\%$	1.05
Left turns	$(10-11)=-1\%$	0.99
Parking not restricted at intersection (See adjustment I-4-C)	$-(0+11)\frac{1}{4}=-3\%$	0.97
Total factor $=0.98\times 1.05\times 0.99\times 0.97=$		0.99

Possible capacity $=1.10\times 0.99\times\frac{40}{70}\times 1{,}380=$ 860 vehicles per hour.

Practical capacity $=0.90\times 0.99\times\frac{40}{70}\times$ 1,380 = 700 vehicles per hour.

East Approach

For average conditions, reported capacity is 1,380 vehicles per hour of green, as shown above.

Adjustments:

Cause	*Effect*	*Factor*
Commercial vehicles	$(10-12)=-2\%$	0.98
Right turns	$(10-24)\frac{1}{2}=-7\%$	0.93
Left turns	$(10-0)=+10\%$	1.10
Parking not restricted at intersection	-6%	0.94
Total factor $=0.98\times 0.93\times 1.10\times 0.94=$		0.94

Possible capacity $=1.10\times 0.94\times\frac{40}{70}\times 1{,}380=$ 810 vehicles per hour.

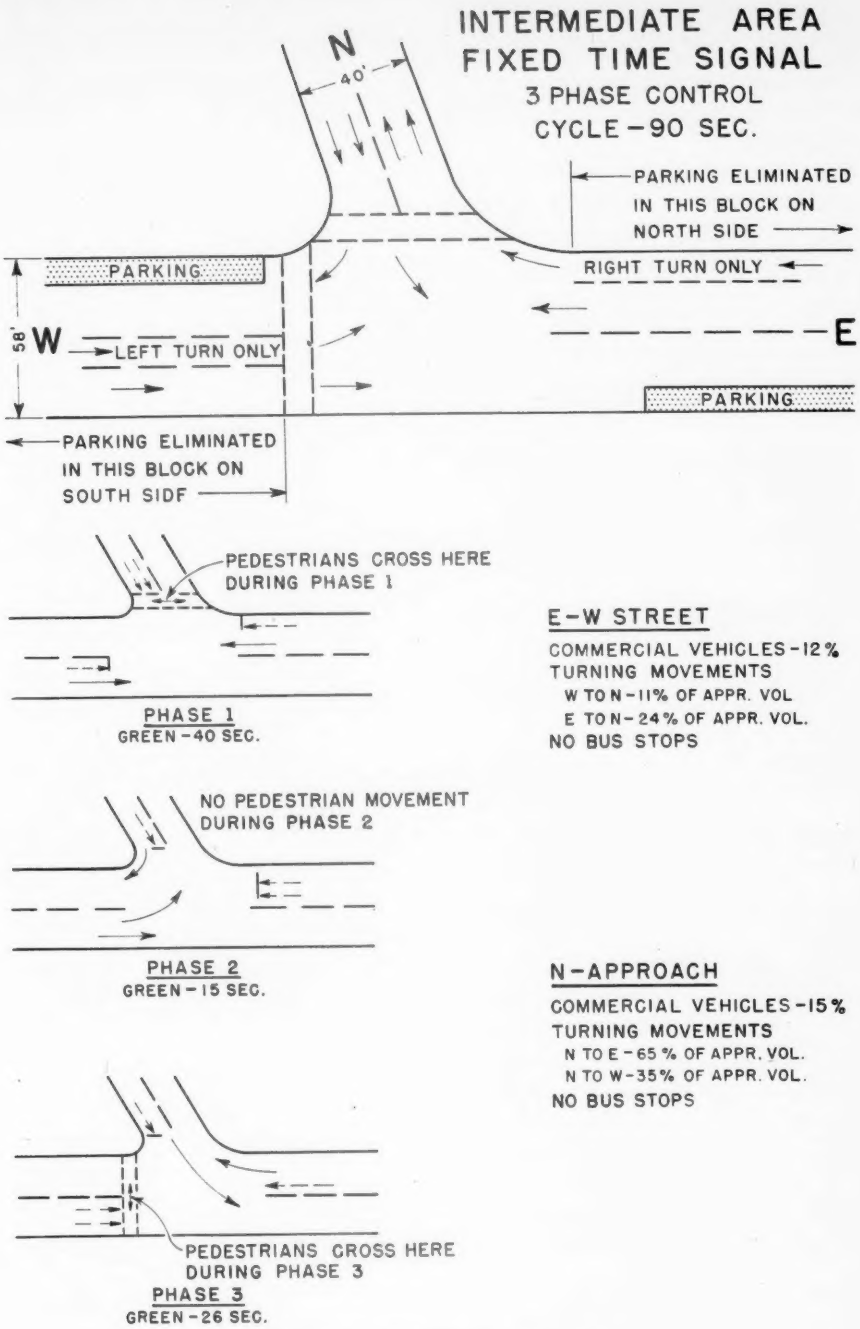


Figure 30.—Illustrative example 6.

Practical capacity $=0.90\times 0.94\times\frac{40}{70}\times$ 1,380 = 665 vehicles per hour.

North Approach

On the intercepted street of a T intersection, the larger of the two turning movements is considered as the through movement; therefore, as indicated on figure 29, the through movement is 65 percent and the turning movement is 35 percent of the total flow during the peak hour.

From figure 24, the reported capacity for a 40-foot street under average conditions is 1,350 vehicles per hour of green.

Adjustments:

Cause	*Effect*	*Factor*
Commercial vehicles	$(10-15)=-5\%$	0.95
Right turns (maximum deduction)	-10%	0.90
Left turns (none)	$(10-0)=+10\%$	1.10
No parking and no bus stops	$+5\%$	1.05
Total factor $=0.95\times 0.90\times 1.10\times 1.05=$		0.99

Possible capacity $=1.10\times 0.99\times\frac{25}{70}\times 1{,}350=$ 525 vehicles per hour.

Practical capacity $=0.90\times 0.99\times\frac{25}{70}\times$ 1,350 = 430 vehicles per hour.

The average daily traffic on the east-west street in the block west of the intersection, when operating at possible capacity, is:

$$\left[860+\left(\frac{40}{60}\times 860\right)\right]\times\frac{100}{9}=15{,}920 \text{ vehicles}$$

per day.

Example 6

Problem

What are the possible and practical capacities of each approach of the intersection shown in figure 30? This is the same intersection as in figure 29 except for partial elimination of parking and substitution of multiphase

signal control in order to increase the capacity of the east-west street and to eliminate vehicle-pedestrian conflicts.

Solution

West Approach

Through movement, phases 1 and 2.—Width available for through movement (parking eliminated as shown) $=\frac{58}{2}-10=19$ feet (see adjustment III–B–1). Capacity for average conditions, from figure 24, for a street twice this width and with no parking is 1,300 vehicles per hour of green. The green interval for phases 1 and 2 = 40 + 3 (amber for opposing through movement) + 15 = 58 seconds.

Adjustments:

Cause	*Effect*	*Factor*
Commercial vehicles	(10−12) = − 2%	0.98
Right turns	(10−0)½ = + 5%	1.05
Left turns	(10− 0) = +10%	1.10
No bus stop	+ 5%	1.05
Total factor = 0.98×1.05×1.10×1.05 =		1.19

Possible capacity $=1.10\times1.19\times\frac{58}{90}\times1{,}300=$ 1,100 vehicles per hour.

Practical capacity $=0.90\times1.19\times\frac{58}{90}\times1{,}300$ = 900 vehicles per hour.

Left turning movement, phase 2.—Left turning is 11 percent of the total volume on the west approach, or $1{,}100\times\frac{11}{89}=135$ vehicles per hour when the approach is operating at possible capacity, and $900\times\frac{11}{89}=110$ vehicles per hour when operating at practical capacity.

Capacity of this 10-foot left-turn lane for 15-second separate green indication and with 12 percent commercial vehicles (factor 0.98):

Possible capacity $=1.10\times0.98\times800\times\frac{15}{90}=$ 145 vehicles per hour (see adjustment III–B–2).

Practical capacity $=0.90\times0.98\times800\times\frac{15}{90}=$ 120 vehicles per hour.

Volumes of 135 and 110, above, therefore do not exceed the capacity of the left-turning lane.

Total capacity of west approach—

Possible capacity = 1,100 + 135 = 1,235 vehicles per hour.

Practical capacity = 900 + 110 = 1,010 vehicles per hour.

East Approach

Through movement, phase 1.—Same as for west approach, except for length of green interval, which is 40 seconds:

Possible capacity $=1{,}100\times\frac{40}{58}=760$ vehicles per hour.

Practical capacity $=900\times\frac{40}{58}=620$ vehicles per hour.

Right-turning movement, phase 3.—Right turning is 24 percent of the total volume on the east approach $=760\times\frac{24}{76}=240$ vehicles per hour when the approach is operating at possible capacity, and $620\times\frac{24}{76}=200$ vehicles per hour when operating at practical capacity.

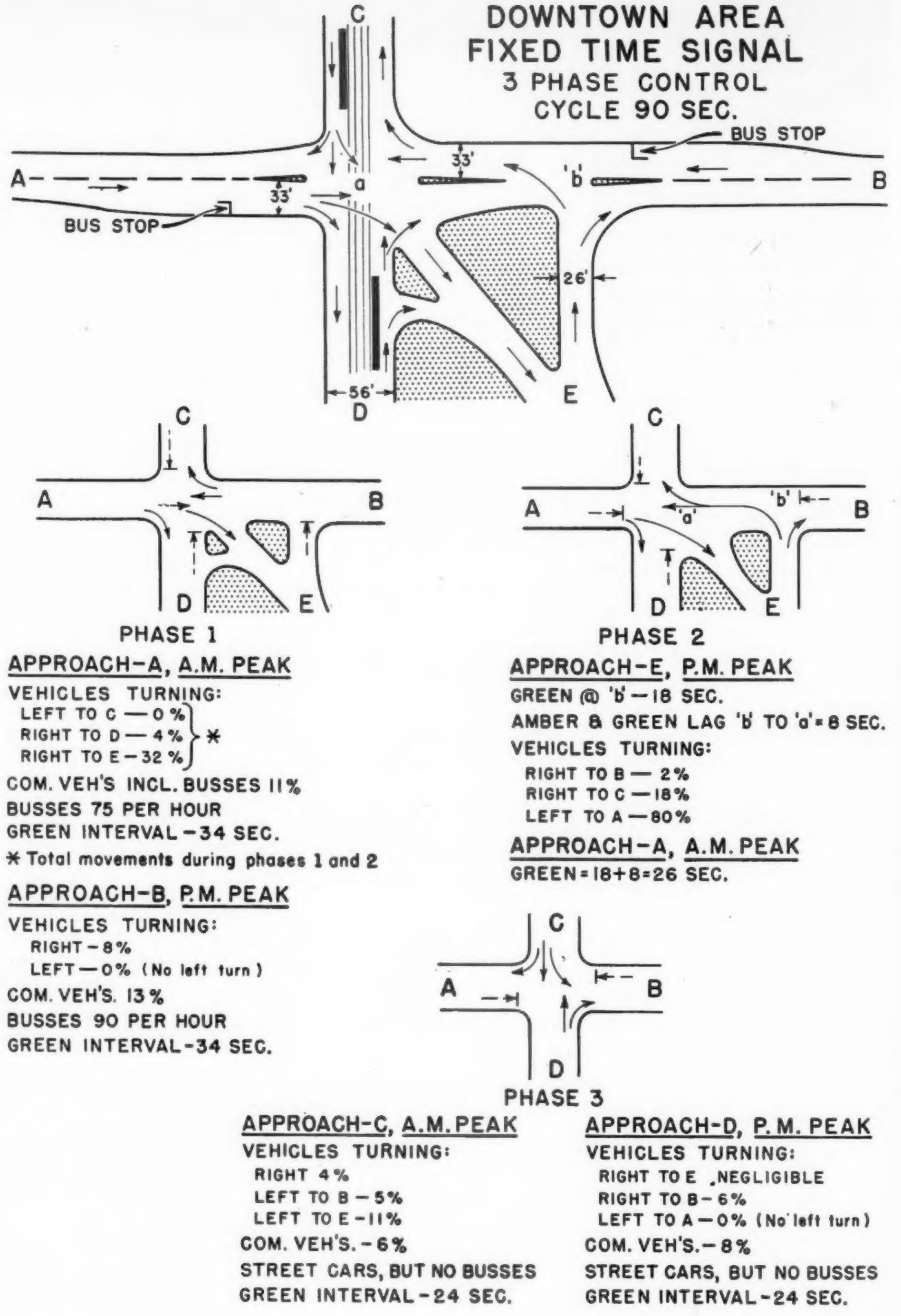


Figure 31.—Illustrative example 7.

Capacity of the 10-foot right-turn lane for 26-second separate green indication and with 12 percent commercial vehicles:

Possible capacity $=1.10\times0.98\times800\times\frac{26}{90}=$ 250 vehicles per hour.

Practical capacity $=0.90\times0.98\times800\times\frac{26}{90}=$ 210 vehicles per hour.

Volumes of 240 and 200, above, therefore do not exceed the capacity of the right-turn lane.

Total capacity of east approach.—

Possible capacity = 760 + 240 = 1,000 vehicles per hour.

Practical capacity = 620 + 200 = 820 vehicles per hour.

North Approach

Left-turning movement, phase 3.—Capacity of the 10-foot lane with 15 percent trucks and 26-second green interval:

Possible capacity $=1.10\times0.95\times800\times\frac{26}{90}=$ 240 vehicles per hour.

Practical capacity $=0.90\times0.95\times800\times\frac{26}{90}=$ 200 vehicles per hour.

Right-turning movement, phase 2.—Right turning is 35 percent of the total volume on the north approach, or $240\times\frac{35}{65}=130$ vehicles per hour when the approach is operat-

ing at possible capacity, or $200\times\frac{35}{65}=110$ vehicles per hour when operating at practical capacity.

Capacity of this 10-foot right-turn lane for 15-second separate green indication and with 15 percent commercial vehicles:

Possible capacity $=1.10\times0.95\times800\times\frac{15}{90}=$ 140 vehicles per hour.

Practical capacity $=0.90\times0.95\times800\times\frac{15}{90}=$ 115 vehicles per hour.

Volumes of 130 and 110, above, therefore do not exceed the capacity of the right-turn lane.

Total capacity of north approach.—

Possible capacity $=240+130=370$ vehicles per hour.

Practical capacity $=200+110=310$ vehicles per hour.

Example 7

Problem

What is the possible capacity of each approach of the multiple intersection shown in figure 31?

Solution

Approach *A*

Phase 1.—The effective width of approach $=33-12=21$ feet (12-foot deduction for at least one bus always loading or unloading during peak hour; see adjustment I-4-A-(d).

Capacity of approach for average conditions, from figure 24, for a street $2\times21=42$ feet wide is 1,510 vehicles per hour of green.

Total green interval per cycle for movements *A* to *D* and *A* to *E* is $34+26=60$ seconds (phase 1 plus phase 2).

During phase 1:

A to $D=\frac{34}{60}\times4\%=0.57\times4\%=2.3\%$ of total approach volume.

A to $E=\frac{34}{60}\times32\%=0.57\times32\%=18.2\%$ of total approach volume.

Adjustments:

Cause	*Effect*	*Factor*
Commercial vehicles [14]	$(10-4) = +6\%$	1.06
Right turns [15]	$(10-2.3)\frac{1}{2}= +4\%$	1.04
Left turns	$(10-0) = +10\%$	1.10
Total factor $=1.06\times1.04\times1.10=$		1.21

Possible capacity, phase $1=\left(1.10\times1.21\times\frac{34}{90}\times1{,}510\right)+75$ busses $=835$ vehicles per hour.

Phase 2.—During phase 2:

A to $D=\frac{26}{60}\times4\%=0.43\times4\%=1.7\%$ of total approach volume.

A to $E=\frac{26}{60}\times32\%=0.43\times32\%=13.8\%$ of total approach volume.

Total capacity, C, of approach *A* during phase 1 and phase 2 is composed of the following:

835 vehicles per hour = total, all movement, phase 1.

1.7% C = *A* to *D* movement, phase 2.

13.8% C = *A* to *E* movement, phase 2.

Then: $C=835+0.017C+0.138C$. Solving, $C=990$ vehicles per hour.

During phase 2:

A to $D=0.017\times990=$ 17 vehicles per hour.
A to $E=0.138\times990=$ 136 vehicles per hour.
Total .... 153 vehicles per hour.

The total, and the movement from *A* to *E*, are less than the capacity of a turning lane (see adjustment III-A-(2).

Approach *E*

Because the street is actually two-way, with a channelizing island, a capacity value from figure 24 is applicable, rather than a value from figure 26, which is for one-way streets.

At point *b* consider left-turn movement to *A* as a through movement. Capacity of approach for average conditions from figure 24 for a street of $2\times26=52$ feet is 1,930 vehicles per hour of green.

Adjustments:

Cause	*Effect*	*Factor*
Commercial vehicles	$(10-10) = 0\%$	1.00
(Since commercial vehicles were not reported, use average conditions)		
Right turns	$(10-2)\frac{1}{2}=+4\%$	1.04
Left turns	$(10-0) = +10\%$	1.10
No bus stop	$+5\%$	1.05
Total factor $=1.00\times1.04\times1.10\times1.05=$		1.20

Possible capacity $=1.10\times1.20\times\frac{18}{90}\times1{,}930=$ 510 vehicles per hour.

At point *a* computations reveal a higher capacity. Point *b* therefore governs.

Approach *B*

Phase 1.—The effective width and capacity for average conditions are the same as for approach *A*.

Adjustments:

Cause	*Effect*	*Factor*
Commercial vehicles [16]	$(10-2) =+8\%$	1.08
Right turns	$(10-8)\frac{1}{2}=+1\%$	1.01
Left turns	$(10-0) =+10\%$	1.10
Total factor $=1.08\times1.01\times1.10=$		1.20

Possible capacity, phase $1=(1.10\times1.20\times\frac{34}{90}\times1{,}510)+90$ busses $=845$ vehicles per hour.

Phase 2.—From approach *E* (above), possible capacity $=510-(0.02\times510)=500$ vehicles per hour.[17]

Phase 1 plus phase 2.—The possible capacity of approach $B=845+500=1{,}345$ vehicles per hour.

Approach *C*

The capacity of the approach under average conditions for a street width of 56 feet with streetcars, from figure 24, is 1,100 vehicles per hour of green.

Adjustments:

Cause	*Effect*	*Factor*
Commercial vehicles	$(10-6) =+4\%$	1.04
Right turns	$(10-4)\frac{1}{2} =+3\%$	1.03
Left turns	$10-(10+6)=-6\%$	0.94
Total factor $=1.04\times1.03\times0.94=$		1.01

[14] Within the limits of accuracy of the solution it may be assumed that busses (75 in number) comprise about 7 percent of the total approach volume. Since commercial vehicles, including busses, comprise 11 percent of the approach volume it follows that 4 percent of this traffic is made up of trucks, for which adjustment must be made. No adjustment is made for busses as commercial vehicles where the 12-foot deduction in street width is made: see adjustment I-4-A-(d).

[15] Movement *A* to *E* is considered as part of the through movement.

[16] Busses are assumed to be 11 percent of the total traffic approaching from *B*. Other commercial traffic is therefore 2 percent of the total: see footnote 14.

[17] The deduction made is for right turn *E* to *B*.

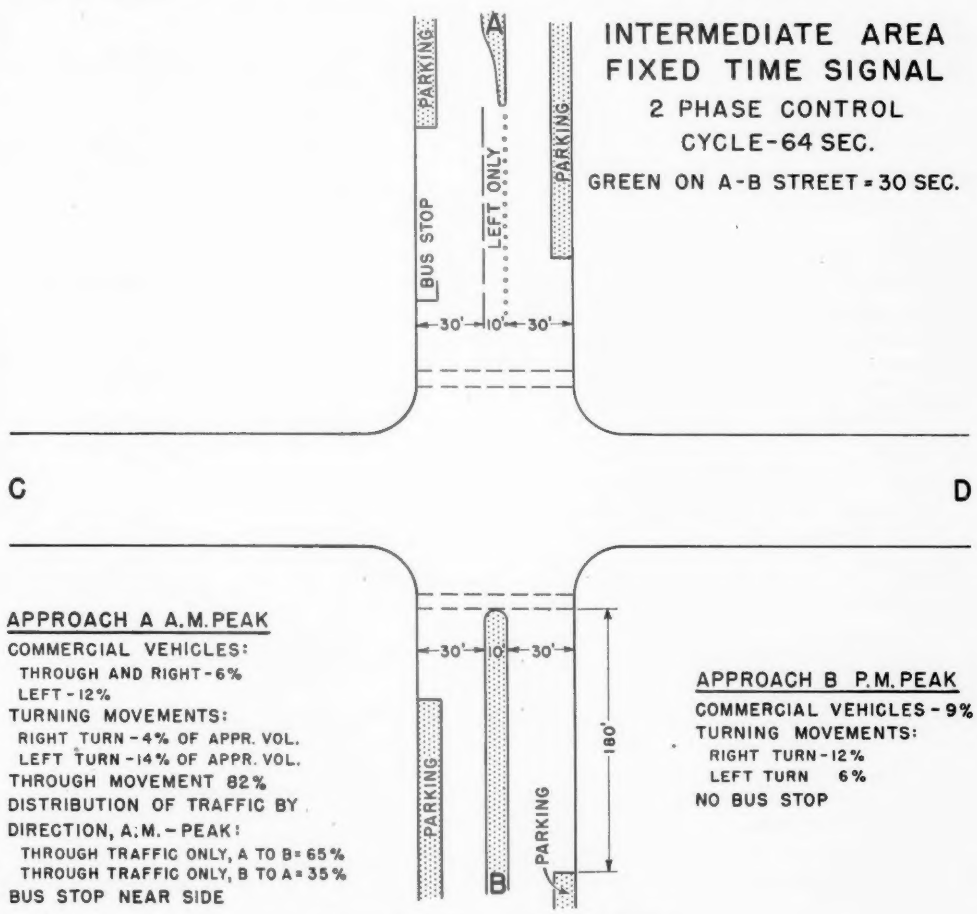


Figure 32.—Illustrative example 8.

Possible capacity $=1.10\times1.01\times\frac{24}{90}\times1{,}100=$ 325 vehicles per hour.

APPROACH *D*

As for approach *C*, the capacity for average conditions is 1,100 vehicles per hour of green.

Adjustments:

Cause	*Effect*	*Factor*
Commercial vehicles	(10−8) =+2%	1.02
Right turns	(10−6)½=+2%	1.02
Left turns	(10−0) =+10%	1.10
Total factor=1.02×1.02×1.10=		1.15

Possible capacity $=1.10\times1.15\times\frac{24}{90}\times1{,}100=$ 370 vehicles per hour.

Example 8

Problem

What are the practical capacities of approaches *A* and *B* of the intersection shown in figure 32?

Solution

APPROACH *A*

Through and right-turn movements.—The reported capacity, from figure 24, for one approach on a street $30\times2=60$ feet wide with parking is 1,430 vehicles per hour of green (the left-turn lane is considered separately below).

Adjustments:

Cause	*Effect*	*Factor*
Commercial vehicles	(10−6) =+ 4%	1.04
Right turns	(10−4)½=+ 3%	1.03
Left turns	(10−0) =+10%	1.10
(Left turns on added turning lane: See adjustment II-2)		
Bus stop	(4+0)¼=+ 1%	1.01
(See adjustment I-4-B-(a): Left turns are considered as zero in this computation because they are made from an added turning lane.)		
Total factor=1.04×1.03×1.10×1.01=		1.19

Practical capacity $=0.90\times1.19\times\frac{30}{64}\times1{,}430=$ 720 vehicles per hour.

Left-turn movement only.—Left turns are 14 percent of the total approach volume, or $720\times\frac{14}{86}=117$ vehicles per hour; or $117\times\frac{64}{30}=$ 250 vehicles per hour of green. With 12 percent commercial vehicles, this is equal to 280 equivalent passenger cars per hour of green.

Check for capacity of left-turn lane.—The opposing through movement, *B* to *A*, during the morning peak is 35 percent of the combined through movement in both directions:

Through movement *A* to $B=720\times\frac{82}{(100-14)}=685$ vehicles per hour.

Through movement *B* to $A=685\times\frac{35}{65}=370$ vehicles per hour, or $370\times\frac{64}{30}=790$ vehicles per hour of green.

With 9 percent commercial vehicles, this is equal to 860 equivalent passenger cars per hour of green.

The capacity of the left-turn lane (see adjustment II-3-(b) is $1{,}200-860=340$ vehicles per hour of green. The left-turn movement of 280 passenger cars per hour of green, or 117 vehicles per hour, can therefore be accommodated.

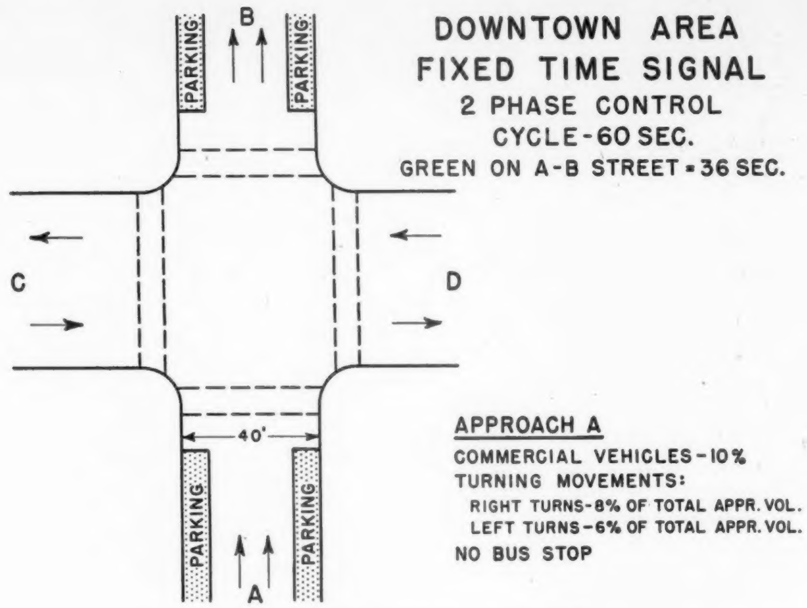


Figure 33.—Illustrative example 9.

Total practical capacity of approach A.—$720+117=837$ vehicles per hour.

APPROACH *B*

The capacity of approach *B*, from figure 24, is the same as that for approach *A*, or 1,430 vehicles per hour of green.

Adjustments:

Cause	*Effect*		*Factor*
Commercial vehicles	(10−9)=	+1%	1.01
Right turns	(10−12)½=	−1%	.99
Left turns	(10−6)=	+4%	1.04
Parking restriction	(12+6)−¼(12+6)=	+14%	1.14
(See adjustment I-4-C)			
Total factor=1.01×0.99×1.04×1.14=			1.19

Practical capacity $=0.90\times1.19\times\frac{30}{64}\times1{,}430$ $=715$ vehicles per hour.

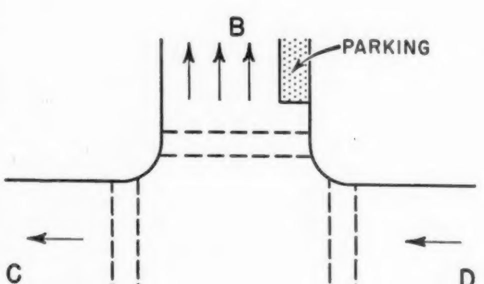


Figure 34.—Illustrative example 10: information as in figure 33, except for parking on right side only with 120-foot restriction.

Example 9

Problem

What is the practical capacity of the one-way street in figure 33?

Solution

From figure 26, the capacity when conditions are average is 1,620 vehicles per hour of green.

Adjustments:

Cause	*Effect*	*Factor*
Commercial vehicles	(10−10) =0%	1.00
Right and left turns	(20−14)½=+3%	1.03
Parking not restricted at intersection	−¼×14 =−4%	0.96
(See adjustment I-4-C)		
Total factor=1.00×1.03×0.96=		0.99

Practical capacity $=0.90\times0.99\times\frac{36}{60}\times1{,}620$ $=870$ vehicles per hour.

Example 10

Problem

What is the practical capacity of the street described in example 9, if parking is eliminated on the left and restricted 120 feet in advance of the cross walk on the right, as shown in figure 34?

Solution

From figure 26, the capacity when conditions are average is 2,250 vehicles per hour of green.

Adjustments:

Cause	*Effect*	*Factor*
Right and left turns	(20−14)½ =+3%	1.03
120-foot parking restriction	$(-\frac{1}{4}\times14)+14\left(\frac{120-20}{5\times36}\right)$	
(See adjustment I-4-C)	=+4%	1.04
Total factor=1.03×1.04=		1.07

Practical capacity $=0.90\times1.07\times\frac{36}{60}\times2{,}250$ $=1{,}300$ vehicles per hour.

Example 11

Problem

What is the practical capacity of each intersection approach on the expressway shown in figure 35?

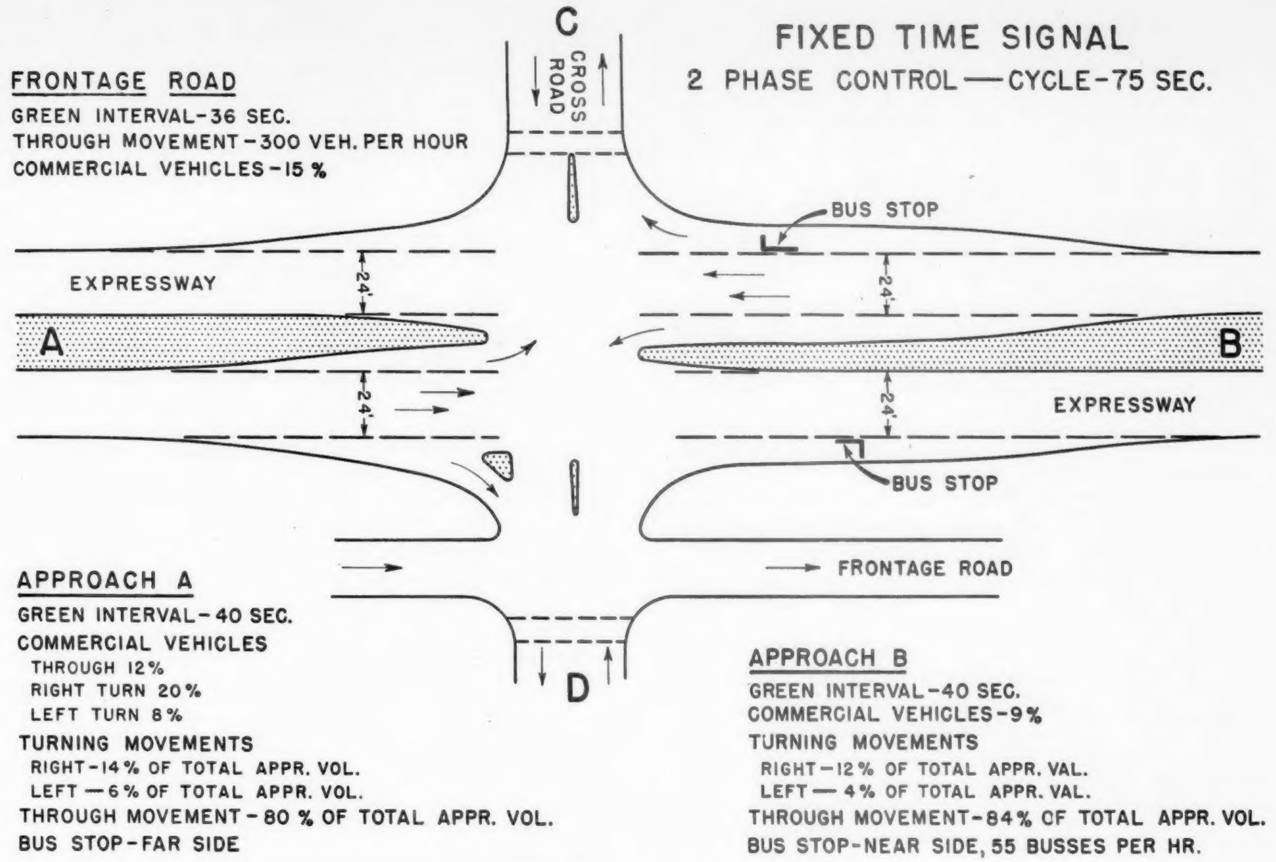


Figure 35.—Illustrative example 11.

Solution

APPROACH *A*

Through movement.— $1{,}000\times\frac{12}{10}\times2=2{,}400$ passenger cars per hour of green (see adjustment V). With 12 percent commercial vehicles, $2{,}400\times0.88=2{,}110$ vehicles per hour of green; or $2{,}110\times\frac{40}{75}=1{,}125$ vehicles per hour.

Right-turn movement.— $2{,}110\times\frac{14\%}{80\%}=370$ vehicles per hour of green, or $370-(0.20\times370)+2\times0.20\times370=445$ equivalent passenger cars per hour of green.

The capacity of the right-turn lane: Volume on frontage road=300 vehicles per hour, or $300\times\frac{75}{36}=625$ vehicles per hour of green, or $625+(0.15\times625)=720$ equivalent passenger cars per hour of green.

Right-turn capacity $=1{,}200-720=480$ passenger cars per hour of green, which is greater than the volume above.

Right-turn movement $=370\times\frac{40}{75}=200$ vehicles per hour.

Left-turn movement.— $2{,}110\times\frac{6\%}{80\%}=160$ vehicles per hour of green, or $160\times\frac{40}{75}=85$ vehicles per hour.

Capacity of left-turn lane (not less than two vehicles per cycle) $=2\times\frac{3{,}600}{75}=96$ vehicles per hour.

(NOTE.—The added turning lane should have sufficient storage capacity to accommodate four standing vehicles clear of the through lanes.)

Total capacity of approach A.— $1{,}125+200+85=1{,}410$ vehicles per hour.

APPROACH *B*

Through movement.—

Adjustments:

Cause	*Effect*	*Factor*
Bus stop (See adjustment V-3-B-2)	$-(12\times\frac{1}{2})=-6\%$	0.94
Commercial vehicles	$-(9\times1)=\ -9\%$	0.91
Total factor=0.94×0.91=		0.855

Practical capacity, less busses $=2{,}400\times0.855\times\frac{40}{75}=1{,}095$ vehicles per hour.

Total practical capacity $=1{,}095+55$ busses $=1{,}150$ vehicles per hour.

Right-turn movement.— $1{,}150\times\frac{12\%}{84\%}=165$ vehicles per hour.

Left-turn movement.— $1{,}150\times\frac{4\%}{84\%}=55$ vehicles per hour.

Total capacity of approach B.— $1{,}150+165+55=1{,}270$ vehicles per hour.

Example 12

Problem

An urban expressway in rolling terrain has two 12-foot lanes in each direction, and all cross streets are separated in grade except at one isolated intersection. At this intersection, calculations indicate that the volume of traffic on the cross street can be accommodated by a two-phase traffic signal if 30 percent of the elapsed time is allowed for movement on the cross street.

The problem is to determine the minimum number of lanes in each direction on the expressway, at the intersection, that will enable the intersection approaches to accommodate a volume of traffic equal to the capacity of the two 12-foot lanes where flow is uninterrupted. Turning movements from the expressway in either direction are: Right turns, 14 percent of approach volume; left turns, 5 percent of approach volume (see fig. 36).

Solution

A 65-second signal cycle is assumed. (Any other length of cycle, within reasonable limits, would be equally satisfactory provided that the shortest green interval is at least 20 seconds.) For the cross street, the green time will be 30 percent of 65, or 19.5 seconds, which may be rounded to 20 seconds. Allowing 5 seconds for amber periods, there remain 40 seconds of each cycle for GO time on the expressway.

The practical capacity of the expressway under free-flowing conditions, with the prevailing number of commercial vehicles and in consideration of the nature of the terrain, is calculated as 1,100 mixed vehicles per lane per hour, or a total of 2,200 vehicles in each direction.

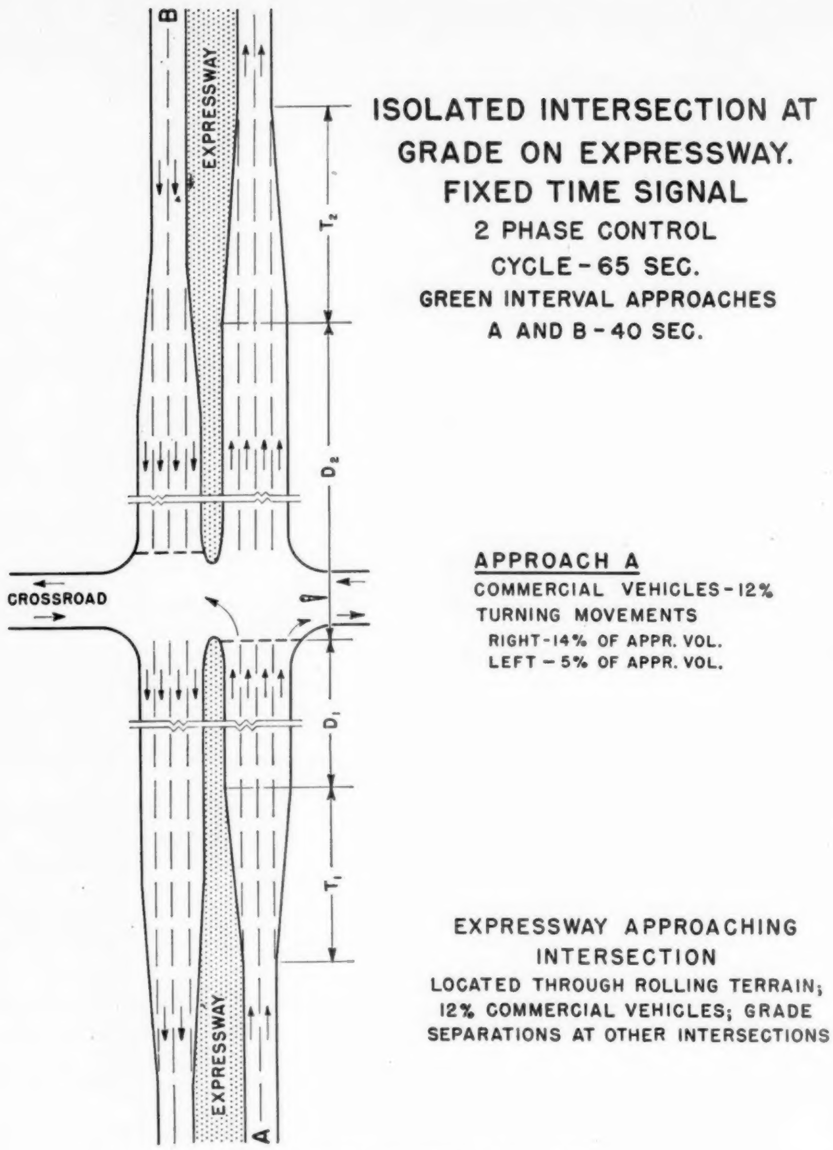


Figure 36.—Illustrative example 12.

The capacity per 12-foot lane at the intersection is 1,200 vehicles per hour of green less necessary adjustments. It should be noted that adjustments for turning movements are necessary (½ percent for right turns and 1 percent for left turns) because the lanes used by turning vehicles are also used by through traffic.

Adjustments:

Cause	*Effect*	*Factor*
Commercial vehicles	12×1 = −12%	0.88
Right turns	14×½ = − 7%	0.93
Left turns	5×1 = − 5%	0.95
Total factor = 0.88×0.93×0.95 =		0.78

Intersection capacity per lane (average) = $1{,}200 \times 0.78 \times \frac{40}{65} = 575$ vehicles per hour.

Number of lanes required in each direction = $\frac{2{,}200}{575} = 3.8$ lanes.

Four lanes, therefore, will be required in each direction at the intersection to accommodate a volume equal to the uninterrupted capacity flow of the expressway. It will be necessary that all lanes be continued through and beyond the intersection in the manner shown in figure 36 because the added lanes would be used by through traffic as well as by turning traffic.

The length of each added lane in advance of the intersection, in feet, (D_1 in fig. 36) should be not less than five times the green interval, in seconds, plus a gradual taper (T_1 in fig. 36). The combined length $D_1 + T_1$ should be sufficient to permit vehicles to decelerate to a complete stop from a normal operating speed on the expressway within this length.

The length of each added lane beyond the intersection (D_2 in fig. 36) should be one and one-half times the distance D_1, and the combined length $D_2 + T_2$ should be sufficiently great to permit vehicles to accelerate from a standstill to the operating speed of traffic on the expressway.

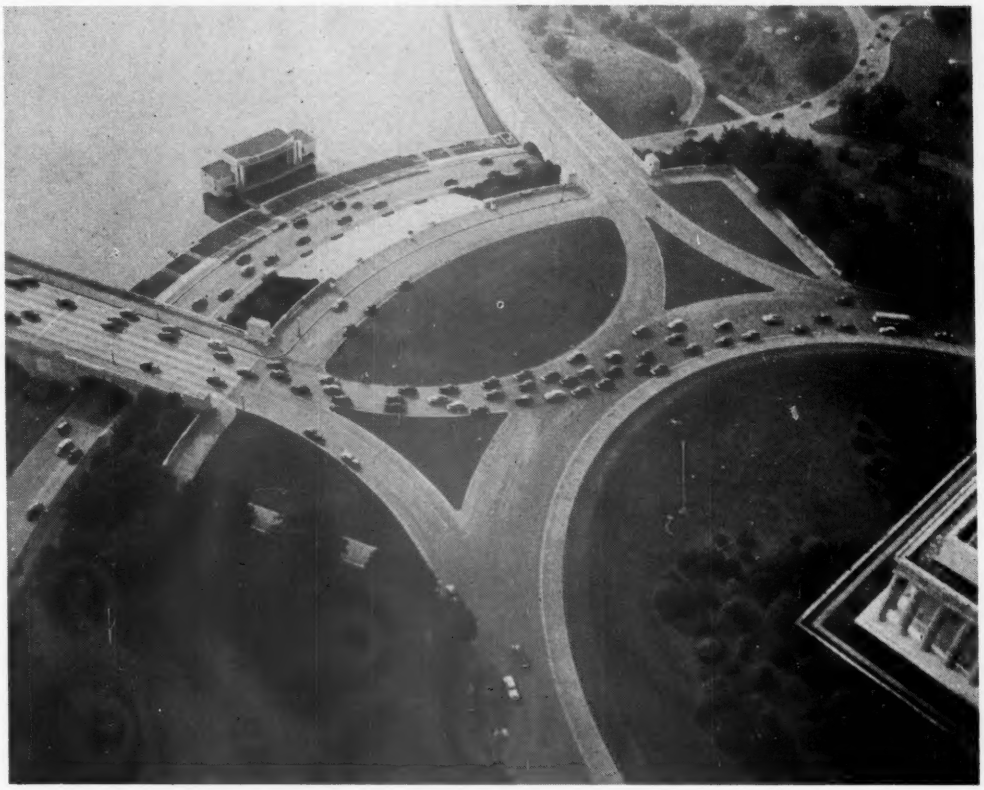
Part VI.—Weaving Sections; Unsignalized Cross Movements

Introduction

Weaving sections are provided for one purpose—to permit the crossing at grade of vehicle pathways with the least possible interference between vehicles. Weaving sections are usually selected as a compromise between the conventional intersection at grade, where delays are often excessive, and the grade separation, with its costly structure and appurtenances. Weaving sections are often provided as adjuncts to grade separations. The traffic circle is, in actuality, a series of weaving sections, and there are many other applications of the principle in the lay-out of controlled-access highways. **Unless consideration is given to that volume of traffic which must cross the path of other vehicles in reaching its destination, the capacity of these higher type facilities may very easily be overestimated.**

Two Classes of Traffic

The vehicles using a weaving section fall logically into two classes: (1) Those entering, passing through, and leaving the section without crossing the normal path of other vehicles, and (2) those that must cross the paths of other vehicles after entering the section. The latter group are the weaving vehicles that make the facility necessary. Consideration of both types of traffic is essential in a study of capacity, but an understanding of the capacity of a weaving section is simplified if the behavior of each class of traffic is dealt with separately. On a well-designed facility operating below capacity, the two classes will actually separate themselves one from the other almost as positively as they do in theory.



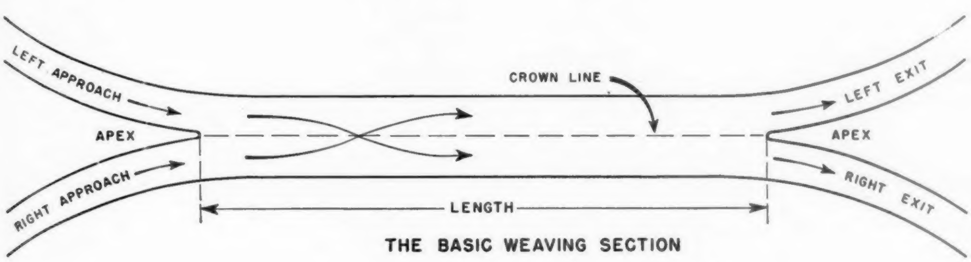
A portion of a traffic circle, showing weaving sections. Rerouting of traffic on some of the approaches to this circle has been resorted to as a means of reducing the conflicting cross movement at this point.

Behavior of Weaving Vehicles

If all the vehicles entering a weaving section from either approach are destined to cross the path of all vehicles entering from the other approach—that is, if all traffic is weaving traffic—it is obvious that every car must cross the crown line somewhere between its extremities (see fig. 37). **At no instant can the number of vehicles in the act of crossing this crown line exceed the number than can crowd into a single lane;** provided, of course, that the facility is operating as it should without vehicles being required to come to a stop before merging with the stream of traffic from the other approach. **Thus, the total number of vehicles entering the weaving section, if all are weaving vehicles, cannot exceed the capacity of a single lane.** This, simply stated, is one of the rules governing the capacity of a weaving section. There are certain modifications of this rule, as will be brought out later, but for the sake of clarity discussion of these modifications is momentarily deferred.

Speed Limited on Short Sections

One of the elements affecting the capacity of a traffic lane is the speed at which vehicles can or do travel. In the same manner, speed also influences the capacity of a weaving section. It is in this respect that the length of the section plays a part in its capacity. To understand the relation between speed and length of section, let it be imagined for the moment that we have a weaving section of very short length, say 50 or 100 feet.



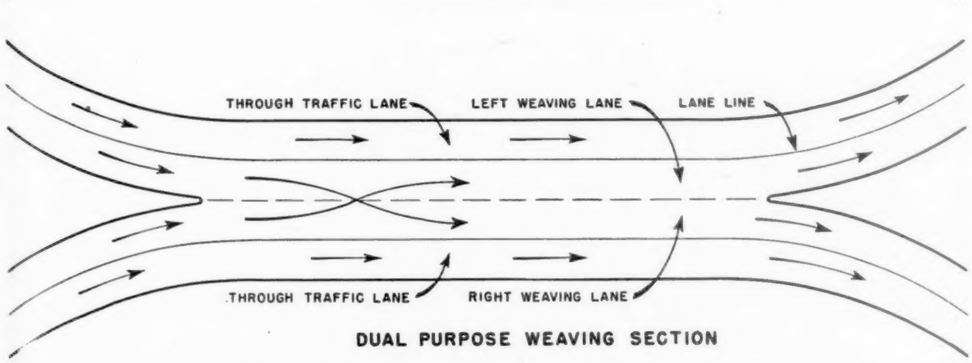


Figure 37* (above).—*The basic weaving section: all traffic weaves.
Figure 38* (below).—*Dual purpose weaving section: serves weaving and nonweaving traffic.

Weaving sections on the Pentagon Network, Arlington, Va.

Further, let it be assumed that traffic is composed entirely of weaving vehicles. At very low traffic volumes there will be little conflict between weaving vehicles even on this short section because gaps in the stream of traffic from one approach will, in most cases, coincide with the entry of vehicles from the other approach. As traffic becomes heavier, however, the probability of vehicles entering the section from the two approaches simultaneously increases until at moderately heavy volumes many drivers will be required to stop and wait for a gap in the other stream of traffic. When the facility is taxed to its capacity, most vehicles will be required to come to a halt and the weaving section fails to serve its intended purpose. Operation is then comparable to that of an ordinary oblique intersection, having a capacity of about 1,200 vehicles per hour. This value corresponds to the possible capacity of a single traffic lane at low speeds.

Under more favorable conditions a weaving section of ample length may have a possible capacity that includes about 1,500 passenger cars per hour performing weaving maneuvers at an average operating speed of about 40 miles per hour. The needed length for this speed is about 900 feet. The same volume of weaving traffic can be accommodated at a speed of 30 miles per hour by a section 450 feet long.

Nonweaving Traffic

Only in rare instances will all traffic be weaving traffic although, as stated earlier, it is for service to the weaving traffic that the weaving section owes its existence. As a secondary purpose, **weaving sections must accommodate the nonweaving traffic by means of added lanes adjoining either side of the weaving lanes** (fig. 38). Determining the capacity of these nonweaving lanes involves no new principle, as the procedure is not different from that for any traffic lane on a multilane facility. **If any weaving section is to function properly and efficiently, it is important that these added lanes have adequate capacity to serve the nonweaving vehicles.** If nonweaving vehicles utilize the weaving lanes either through choice or through necessity, they interfere with the vehicles that must weave to reach their destination, thus reducing the total number of weaving vehicles that can be accommodated. Appropriate use of signs to direct drivers to the proper side of the approach road is essential, therefore, if the section is to operate efficiently during peak volume periods. Also, the effective length of a weaving section is influenced to some extent by the distance ahead of the weaving section that drivers on one approach road can see traffic on the other approach road. This distance may be used by drivers that must cross the paths of other vehicles to adjust their speeds before reaching the weaving section, so that the merging operation will be performed with a minimum of conflict between vehicles.

Typical Examples

Information regarding the relations between traffic volume, operating speed, and geometric features of weaving sections has been obtained from detailed studies conducted by the Bureau of Public Roads and from traffic volume data submitted by State highway departments and Committee members. Table 20 shows data pertinent only to the sections of considerable length which were operating at or near capacity and for which detailed data such as speeds are available.

Location No. 1—Pentagon Network

Weaving section No. 1 lies between two ramps and serves a part of the traffic to and from Arlington Memorial Bridge at Washington, D. C., as well as some traffic from Lee Boulevard destined for the Pentagon Building in Arlington, Va. Trucks are not allowed on this facility and, except for a negligible number of busses, traffic is composed entirely of passenger cars. There are no lane lines or longitudinal joints on the bituminous surface, but both approaches and the exit to the left, which serves as a ramp approaching the bridge, are used as two lanes. The weaving section is of sufficient width to accommodate four lanes of traffic. Figure 39 shows the number of vehicles in each lane on the approach roadways that entered each lane on the exit roadways but does not show the exact points at which the weaving maneuvers took place.

Vehicles from Lee Boulevard, or those approaching the weaving section from the right, traveled at an average speed of about 25 miles per hour. Speeds on the ramp from Memorial Bridge averaged 20 miles per hour with about 5 percent of the vehicles coming to a complete

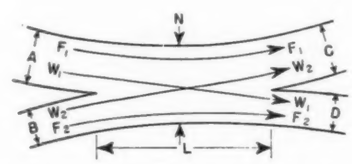


Table 20.—*Observed volumes on weaving sections*

Location number [1]	Dimensions						Traffic volume in vehicles per hour						Approximate speeds
	Length L	Number of lanes at—					W_1	W_2	W_1+W_2	F_1	F_2	Total	
		A	B	N	C	D							
	Feet												*M. p. h.*
1	1,140	2	3	4	2	3	1,496	1,794	3,290	0	246	3,536	25
2	1,146	2	3	[2] 4	2	2	772	1,712	2,484	0	240	2,724	18
3	921	3	3	4	3	2	1,000	309	1,309	915	790	3,014	40
4	488	3	2	3	2	3	1,698	276	1,974	450	348	2,772	22
5	550	3	3	5	2	3	419	1,676	2,095	342	764	3,201	28
5	550	3	3	5	2	3	1,351	384	1,735	450	582	2,667	30
6	550	3	3	5	2	3	945	755	1,690	714	437	2,841	27
6	550	3	3	5	2	3	509	1,672	2,181	638	425	3,244	25

[1] Locations 1–4 are on the Pentagon Network, Arlington, Va.; locations 5 and 6 are on the San Francisco Bay Bridge distribution structure.
[2] Narrows from 44 feet to 27 feet approximately midway of length.

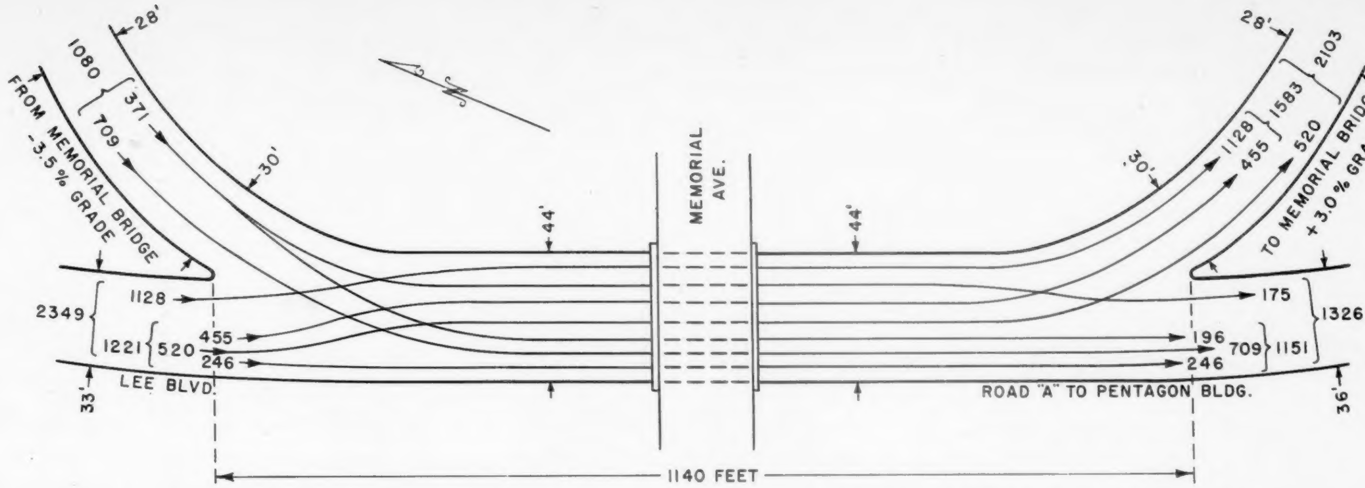


Figure 39.—Weaving section on roadway under Memorial Avenue near west end of Memorial Bridge, Washington, D. C. (morning rush period, 7:45–8:45 a. m., November 13, 1947).

stop before entering the weaving section. Near the exit from the weaving section, vehicles in the left lane were spaced as closely together as possible and all were moving at practically the same speed—15 to 20 miles per hour. The average speed in the right-hand lane near the exit was about 25 miles per hour with 20 percent of the vehicles traveling 30 to 35 miles per hour.

It was apparent during the period of peak volume that the desired number of weaving maneuvers could not be performed in a safe and efficient manner, as evidenced by the 175 vehicles destined to the right-hand exit that were forced to stay in the left lane for the full length of the weaving section before crossing the path of vehicles taking the left-hand exit. Also, it was obviously impossible for all vehicles turning left to Memorial Bridge to enter the left lane, which accommodated 1,583 vehicles in the 1 hour and frequently accommodated vehicles at a rate approaching the basic capacity of 2,000 vehicles per hour for periods of several minutes duration.

Location No. 2—Pentagon Network

Figure 40 shows the details of location No. 2, the same facility as location No. 1, but as it appeared in 1946 prior to improvement to the present lay-out. It will be noted that the improved facility (fig. 39) accommodates about 700 vehicles per hour more than the number observed on its predecessor. This over-all increase consists almost entirely of weaving vehicles. The service rendered to 3,429 vehicles is comparable to that previously rendered to the 2,724 vehicles per hour using the former facility. Since the improvement, there has been a slight increase in speed, and weaving maneuvers can be performed with somewhat greater ease. The present facility provides slightly better service because only 175 vehicles, as compared with 270 on the earlier facility, are forced to travel the full length of the section in the left lane before being able to cross to the exit leading to road *A*. On the old facility, represented by figure 40, nearly all weaving maneuvers were performed on that portion of the section north of Memorial Avenue, whereas on the newer facility, weaving is accomplished throughout the length of the weaving section.

Location No. 3—Pentagon Network

Pertinent physical features and the maximum observed traffic volume at location No. 3 on the Pentagon Network, commonly referred to as the southbound mixing lanes, are shown in figure 41. At this location the two approach routes and the two exit routes are of equal importance so no one of the four traffic movements has been shown any preference. The arrows show only the total volume entering and leaving on each roadway.

Studies have shown that the average speed is 37 miles per hour on the two approach roads, with the following distribution of speeds:

	Percent
Below 30 m.p.h	14
30 to 39 m.p.h	53
40 to 49 m.p.h	30
50 m. p. h. or over	3

Figure 40.—Maximum observed hourly traffic volume on mixing lane section on roadway under Memorial Avenue near west end of Memorial Bridge, Washington, D. C.

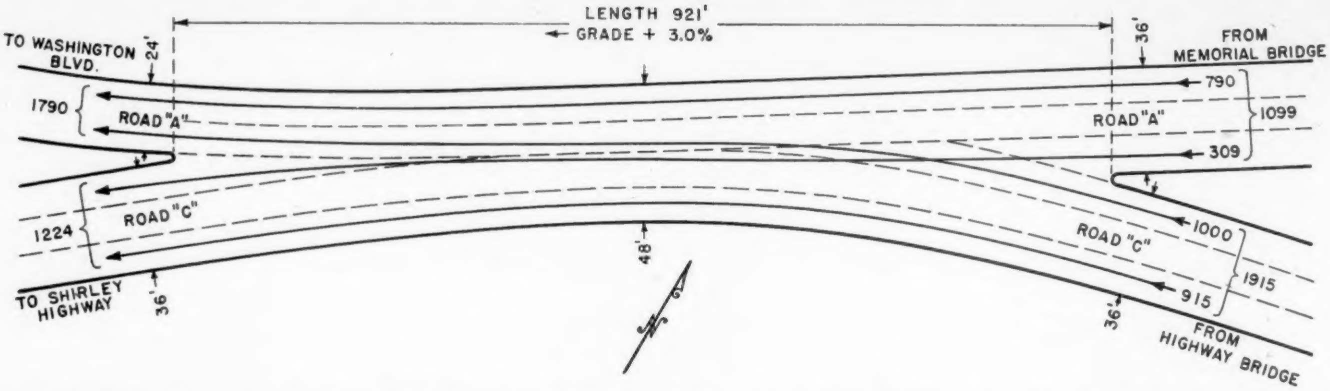


Figure 41.—Maximum observed hourly traffic volume on mixing lanes for south-bound traffic at junction of road **A** *from Memorial Bridge and road* **C** *from U S 1, Arlington, Va.*

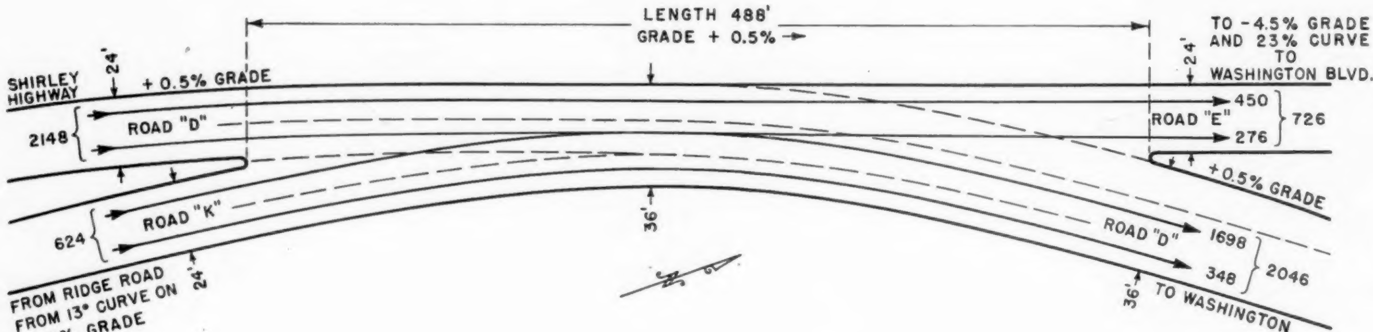
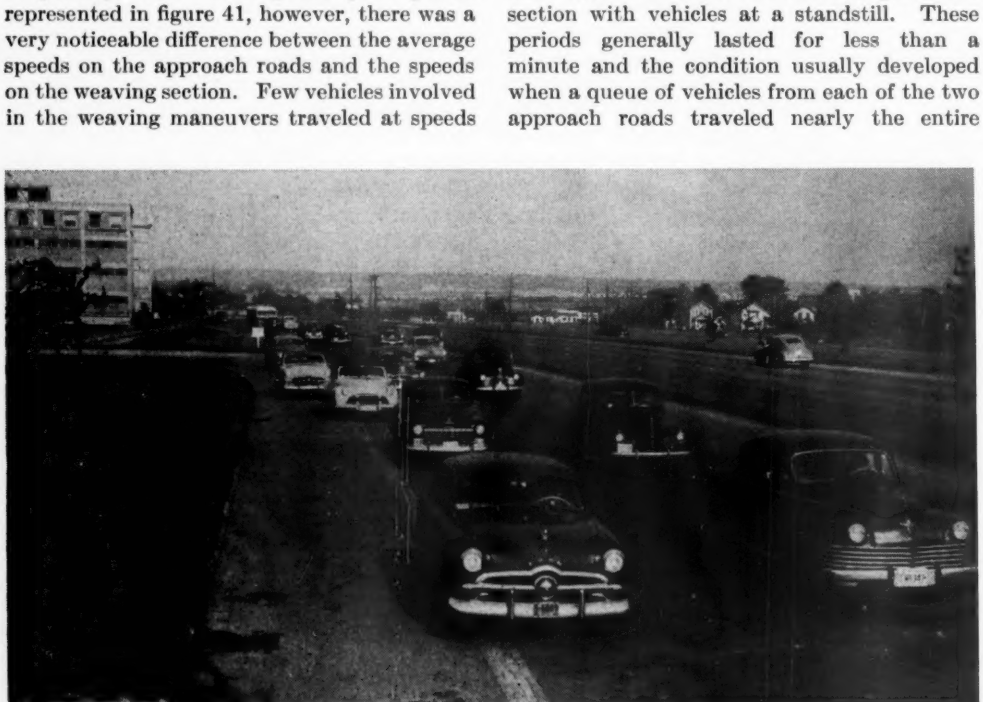


Figure 42.—Maximum observed hourly traffic volume on mixing lanes for north-bound traffic at junction of road **K** *from Ridge Road and the Shirley Highway, Arlington, Va.*

Very few drivers reduce their speeds while on or approaching the weaving section during off-peak periods. During the peak period represented in figure 41, however, there was a very noticeable difference between the average speeds on the approach roads and the speeds on the weaving section. Few vehicles involved in the weaving maneuvers traveled at speeds in excess of 30 miles per hour, and on several occasions during the hour the two weaving lanes became filled for the entire length of the section with vehicles at a standstill. These periods generally lasted for less than a minute and the condition usually developed when a queue of vehicles from each of the two approach roads traveled nearly the entire length of the section at approach road speeds without performing the necessary weaving maneuvers before coming to a complete stop immediately ahead of the apex separating the exit roadways.

During the hour of study, the total flow was 3,014 vehicles, including 309 passenger cars from one approach road that crossed the paths of 1,000 vehicles (4 percent dual-tired trucks and busses) from the other approach road. Although the roadway appeared to have reached its practical capacity, the volume of traffic approaching on the right and leaving on the right, or approaching on the left and leaving on the left, could have been considerably greater without increasing the congestion had these vehicles not become involved with the ones that were weaving. Also, lower approach-road speeds would have permitted a larger number of vehicles to perform the necessary weaving maneuvers within the same length of weaving section without increasing the apparent congestion. This section exceeds the minimum length required for the prevailing volume of traffic, but the added length overcomes in part the deficiency of inadequate signs and the absence of lane lines.



Entrance turn from ramp to freeway. Although traffic from the ramp is rather light, it is sufficient to fill the available spaces in the first lane of the freeway. The lane adjacent to the median is almost devoid of traffic. This characteristic tendency for traffic to distribute itself between lanes results in many of the spaces being shielded against occupancy by vehicles from the ramp.

Location No. 4—Pentagon Network

The traffic movements at location No. 4 on the Pentagon Network are shown by figure 42. In this case, traffic approaching on the Shirley Highway (road *D*) headed toward Washington (also road *D*) has been shown a definite preference by the arrangement of the longitudinal joints, which act as lane lines,

and by the general alinement. Traffic slowed to about 20 miles per hour for short periods during the hour of peak volume. A slight increase in the total volume of weaving traffic would have resulted in very congested conditions.

Weaving Sections Limited in Practical Application

The results of analysis of available data on traffic volumes and speeds on weaving sections are shown by figure 43. Basically, traffic on a weaving section is affected by density in much the same manner as on a roadway with uninterrupted flow. Maximum volumes occur at speeds between 20 and 30 miles per hour. Higher speeds are possible only at volumes and traffic densities lower than those found when the facility is operating at its possible capacity. **Whenever traffic density exceeds the critical density, speeds fall below 20 miles per hour, the capacity is lowered, and complete congestion or stagnation may occur within a few seconds.**

Weaving section for north-bound traffic at the junction of Shirley Highway and Washington Boulevard, Arlington, Va.

The curves in figure 43 show that there is a rapid increase in the length of the section required for a given speed with an increase in the number of weaving vehicles. Doubling the traffic volume approximately triples the length of section required and doubles the number of lanes required for the weaving vehicles.

Figure 44 shows schematically the weaving maneuvers that must take place when the number of weaving vehicles is double the normal capacity of a traffic lane. All vehicles are shown crossing the crown line either in the first one-third or last one-third of the section. Each vehicle is involved in two weaving maneuvers with the result that four times as many weaving maneuvers must be performed as with half the volume. Theoretically, at least, this illustrates the need for tripling the length for twice the volume.

In practice, however, most drivers, knowing in advance that they must cross the crown line, attempt to gain a position on the approach road which is most favorable for the early performance of the weaving maneuver. As a result, the two streams of vehicles that must weave approach each other with no gaps to permit an interchange of positions. **Any weaving section, regardless of its length or number of lanes, will become badly congested when the number of weaving vehicles approaches the possible capacity of two traffic lanes. Operating conditions will seldom be entirely satisfactory unless the traffic on the approach roadways is well below the practical capacities of these approaches and the weaving section has one more lane than would normally be required for the combined traffic from both approaches. For this reason, weaving sections are considered practical only where the two intersecting one-way roadways each carries less than the normal capacity of two lanes of a one-way roadway and the total number of vehicles required to weave does not exceed 1,500 per hour.** In computing capacities, allowance must be made for trucks, lanes of substandard width, etc., on the same basis as in the case of multilane roadways.

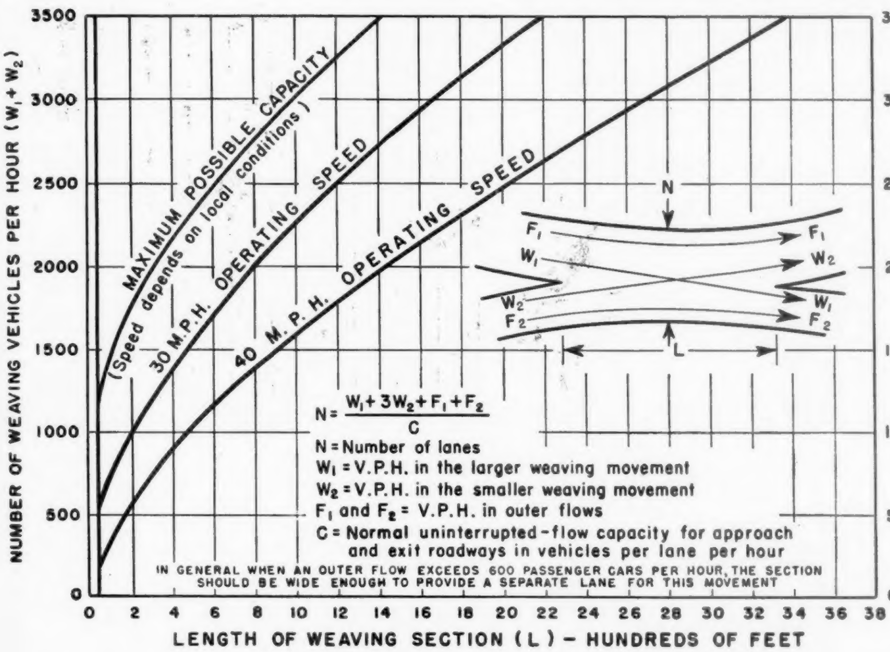


Figure 43.—Operating characteristics of weaving sections.

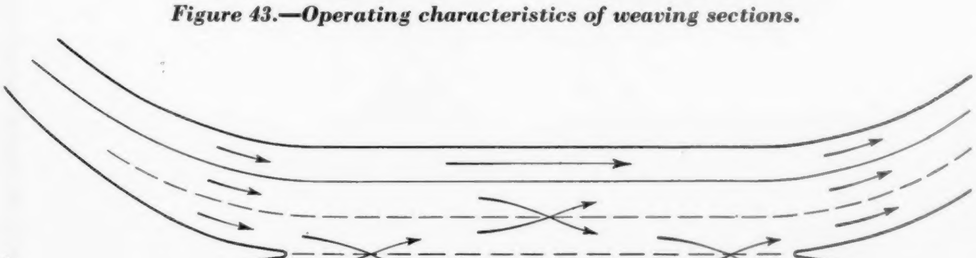


Figure 44.—A compound weaving section.

Part VII.—Ramps and Their Terminals

INTRODUCTION

The efficiency of traffic movement on freeways or expressways and the extent to which their potential capacities can be realized depends directly on the adequacy of the facilities that are provided for entering and leaving these highways. Improperly planned entrances can seriously limit the traffic volumes that can use an expressway, and exit facilities incapable of accommodating vehicles leaving the highway at one point, even though the number be relatively few, can cause complete congestion of all traffic.

While the subject of ramp capacities has been considered by the Committee and much factual data have been collected, only a few generalizations can be drawn at the present time that might be helpful in dealing with problems concerning ramp capacities. No exact rules are applicable to all ramps because their capacities are so closely related to the particular lay-out, especially at the terminals. The primary purpose of this section, therefore, is to furnish some knowledge of traffic operations on ramps that can be utilized by engineers in rationalizing the various elements that should be considered.

The capacity of a ramp is affected by the character of traffic, gradient, width, curvature, and the speed at which vehicles operate. **The sharp curvature on most ramps usually limits the possible capacity to that attained on a tangent section at speeds below 20 miles per hour.** At the higher speeds, drivers find it difficult to stay within their lane and have a tendency to maintain a somewhat greater headway than is normal on tangent sections. This happens despite the fact that lanes wider than 12 feet are often provided on ramps.

Ramps may be of different forms, but the two general classes are inner loops and direct connections.

Except on ramps having extremely long radii, and on direct connections having a low degree of curvature, the lane capacity of the ramp itself, when the entrance or exit does not govern, is usually of the order of 1,200 passenger cars per hour. Thus, a ramp having a nominal width of two lanes (usually 28 or 30 feet) should accommodate about 2,400 passenger cars at an average speed of 12 to 15 miles per hour. Actual examples of ramp movements of this magnitude are rare, indeed, because few of the terminals are so planned that traffic entering and leaving the ramp can do so with the necessary freedom of interference from traffic on the major highways or streets.

Even on some of the most modern facilities, there is a tendency for traffic to move in a single line at one or two points on the ramp. The point of constriction is usually at either the entrance or the exit to the ramp. Under such conditions the possible capacity of the ramp will be about 1,200 vehicles per hour. This probably accounts for the common belief that the capacity of any ramp will not exceed 1,200 to 1,500 vehicles per hour and therefore the ramp should be tapered to confine vehicles on the ramp to one lane as they enter an expressway. **However, there is little reason to think that ramp terminals cannot be so planned that much larger volumes of traffic can be handled with ease.** Further, there are a few existing facilities where much higher volumes have been observed. One such example is the ramp shown as the left exit of the weaving section illustrated in figure 39. Nearly every day about 2,100 vehicles per hour during the morning rush hour use this ramp, which is approximately semicircular in shape and has a radius slightly over 200 feet. The vehicles from the ramp merge with about 1,200 vehicles per hour in three lanes at the ramp terminal.

Another example of an unusually heavily traveled ramp is the Dearborn Street interchange on North Lakeshore Drive in Chicago. At the on-bound ramp 2,092 vehicles in 1 hour have been observed merging with 5,923 vehicles on the expressway. The ramp is a direct connection and traffic merges in six lanes. The off-bound ramp at this interchange has been observed to discharge 2,482 vehicles in 1 hour. This is also a direct connection.

Where direct connections having sufficiently large radii of curvature and superelevations to permit speeds of 30 miles per hour or more during heavy traffic volumes are employed, the practical capacities of the lanes on the ramp and the expressway may be calculated as for any traffic lane in the manner outlined in part IV, providing traffic using the ramp is not hampered in entering and leaving the ramp by through traffic on the highways which the ramp connects. Generally, however, the capacity of a ramp is controlled by conditions at or near the terminals.

CONDITIONS AFFECTING RAMP CAPACITY

There are almost an unlimited number of different conditions and combinations of conditions that limit the number of vehicles that can use a given ramp, or that determine the necessary ramp design for a certain number of vehicles. Only a few of the more important conditions that must be considered are covered

in the following discussion. Any one of these might be the controlling factor for a particular location.

Volumes During Peak Periods

For no portion of a highway is it more important to know the volume of traffic in the various movements during peak periods than when trying to design an adequate ramp or when estimating the capacity of an existing ramp. Annual volumes or 24-hour volumes for the various movements are of little value, because the peak volumes on some ramps do not coincide with the peak volumes on the highways. Oftentimes the peak volume on one ramp will occur during periods when the traffic volumes on other nearby ramps or on the highway are relatively low. For this reason, the traffic volume that can be accommodated by a particular ramp will vary from time to time depending on the traffic volumes using the facilities that it connects.

Weaving Distance Between Ramps

The number of vehicles that can leave or enter a ramp is sometimes controlled by the number of vehicles leaving or entering the same highway on adjacent ramps. In the case of the two inner loops of a cloverleaf design which are used by traffic traveling in the same direction on a main highway, traffic entering the main highway from the on-ramp must weave with traffic entering the off-ramp. For this condition, the capacity of the section of roadway between the two ramps can be calculated in the same manner as for any weaving section. **If the distance between the two inner loops is very short, the combined capacity of the two ramps would be restricted to a total of about 1,200 passenger cars per hour at a speed of less than 20 miles per hour.** This is one reason that designs in which vehicles leave an expressway facility ahead of the point where vehicles enter the facility are preferred to the cloverleaf design, especially when the movements on two adjacent inner loops are at or near their peaks at the same time.

Terminal Conditions

The entrance to a ramp might be made from a facility on which the flow of traffic is uninterrupted, or it might be from a highway or city street with cross traffic at grade and be in the immediate vicinity of an intersection controlled by traffic lights. Likewise, the exit from a ramp might be made either to a facility with uninterrupted flow or to a highway or street with cross traffic at grade. Furthermore, either or both of the roadways which the ramp connects might have acceleration and deceleration areas or added lanes for the vehicles using the ramp. Also, if there are no acceleration areas or added lanes, the general lay-out may be such as to require traffic to stop before entering the highway or street. **There are many different conditions that can exist at the entrance and exit of a ramp, any one of which might control the number of vehicles that can use the particular ramp.**

At-grade intersections in vicinity of ramp

Traffic approaching a ramp leading to an expressway and traffic on a ramp leading from an expressway must oftentimes pass through an intersection at grade in the immediate vicinity of the ramp. **The volume of traffic that can be accommodated by the ramp is then dependent on the capacity of the nearby intersection.** When this is the case, the information on intersection capacities should be applied when estimating the maximum volume of traffic that can enter or leave the ramp.

Entrance to expressway from ramp

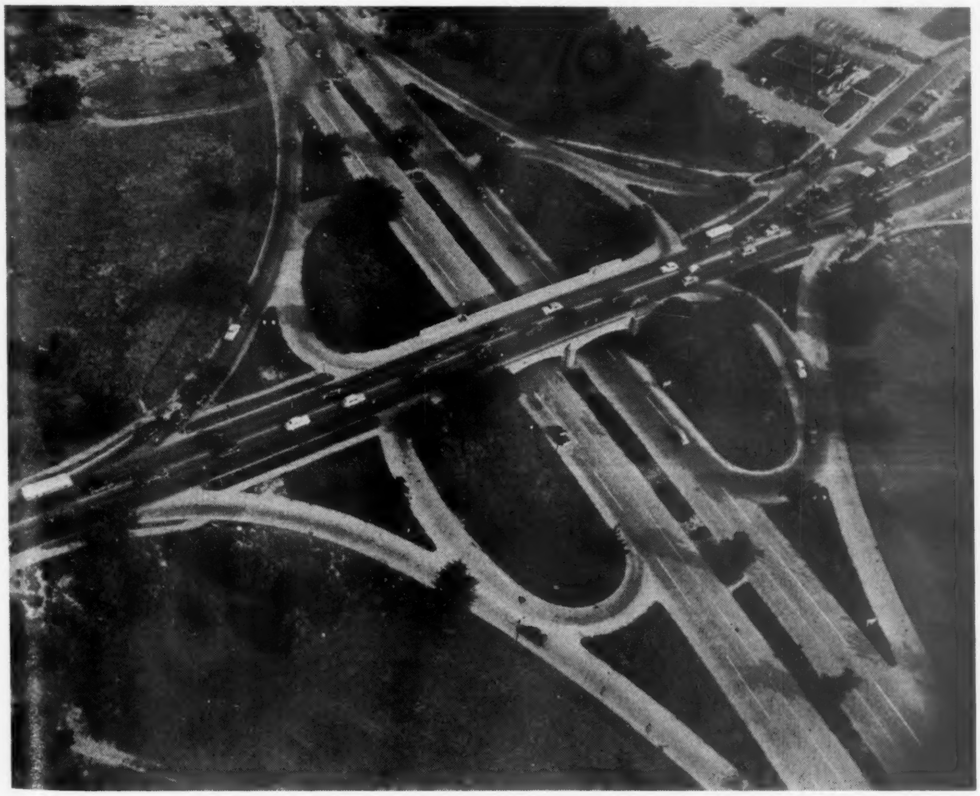
It is a common practice to assume that as many vehicles can enter an expressway from a ramp as the difference between the capacity of the expressway beyond the ramp and the traffic volume on the expressway ahead of the ramp. For example, if a four-lane divided expressway had a practical capacity of 3,000 vehicles per hour in the one direction and there were 1,800 vehicles per hour on the expressway just ahead of the on-ramp, it would normally be assumed that 1,200 vehicles per hour could enter the expressway from the ramp. With no acceleration area, this would be true only if the 1,800 vehicles already on the expressway could and would crowd into the one traffic lane before reaching the on-ramp. It would also be true if there were an acceleration area of the same length as the distance necessary for 3,000 vehicles traveling in three lanes to crowd into two lanes without a marked reduction in speed.

Obviously, vehicles from a ramp can enter an expressway only in the available spaces between vehicles operating in the first lane of the expressway. **The higher the traffic volume on the expressway, the less opportunity there will be for vehicles to enter the expressway when other conditions are comparable.**

The length of the space between vehicles on the expressway that will be utilized by a driver entering from a ramp will vary for different drivers and with the frequency of such spaces and the speed of traffic. If a driver has to wait a long time for the occurrence of a space which he normally considers entirely adequate, he is apt to take advantage of a somewhat shorter space that occurs more frequently.

The results of a study made to determine the spacing between vehicles in the first lane of an expressway that drivers utilize in entering from a ramp on the right are shown by figure 45. These results represent only one particular set of conditions, i. e., a continuous back-log of vehicles on a ramp 20 feet wide with a stop sign strictly enforced and no acceleration area for traffic before entering the through lanes of the expressway. To eliminate or at least reduce the effect of one of the variables—the speed of traffic on the expressway—the spacing between vehicles on the expressway has been expressed as the time interval between vehicles as they approached the location of the ramp.

The upper part of figure 45 shows that some drivers, although very few, entered the expressway from the ramp when the time spacing (center to center) between passenger cars in the first lane of the expressway was as low as 1 second. Other drivers would not enter unless the time spacing was as high as 17 seconds. In other words, with a double line of vehicles on the ramp waiting to enter



A ramp terminal where the traffic demand exceeds the capacity of the expressway. During peak periods traffic must be directed by a police officer.

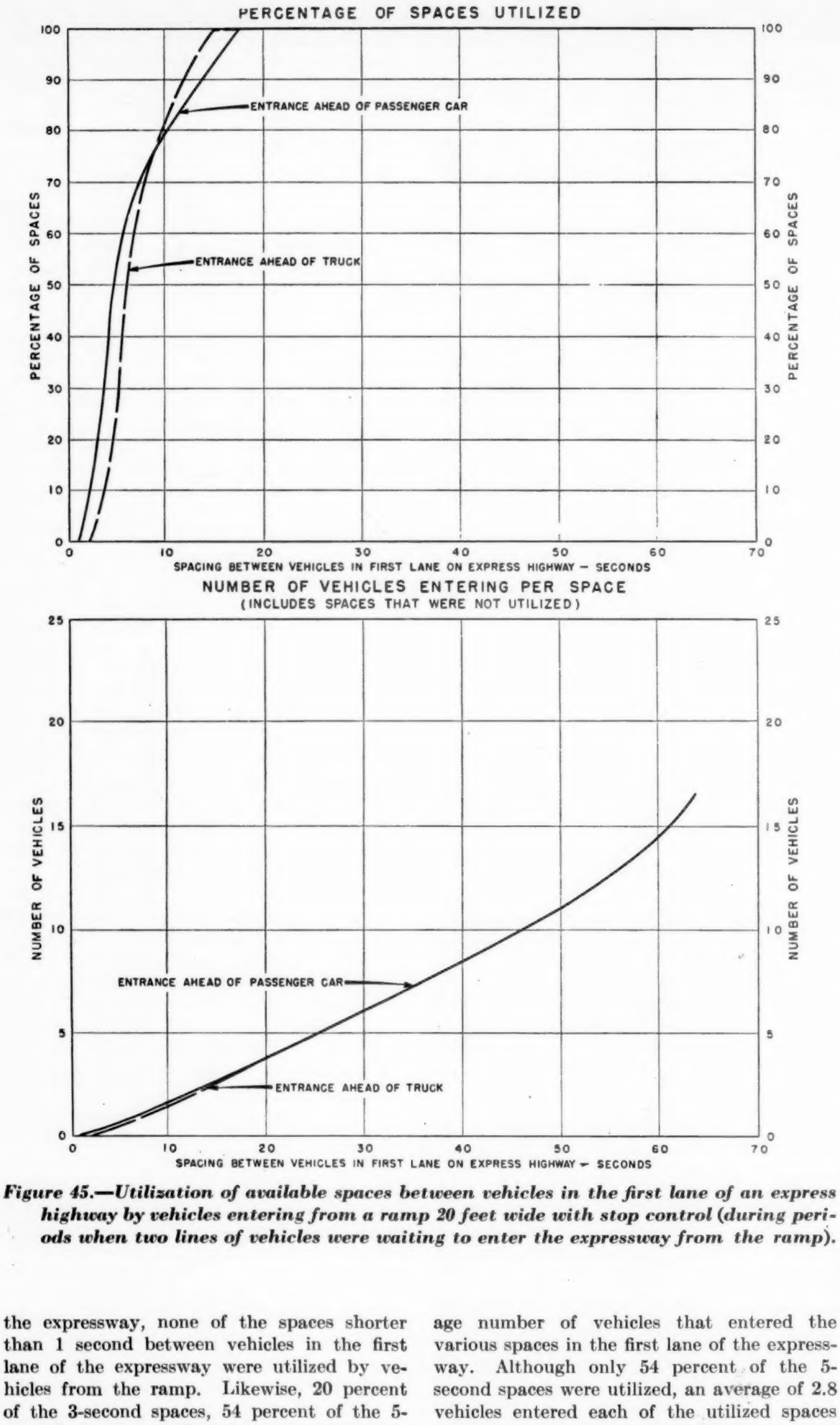


Figure 45.—Utilization of available spaces between vehicles in the first lane of an express highway by vehicles entering from a ramp 20 feet wide with stop control (during periods when two lines of vehicles were waiting to enter the expressway from the ramp).

the expressway, none of the spaces shorter than 1 second between vehicles in the first lane of the expressway were utilized by vehicles from the ramp. Likewise, 20 percent of the 3-second spaces, 54 percent of the 5-second spaces, and all of the spaces 17 seconds or longer were utilized by vehicles in entering the expressway from the ramp. When the space occurred ahead of a truck on the expressway, an entrance from the ramp was never made unless the time space was at least 2 seconds, and at least one vehicle always entered when the time space was 15 seconds.

The lower part of figure 45 shows the average number of vehicles that entered the various spaces in the first lane of the expressway. Although only 54 percent of the 5-second spaces were utilized, an average of 2.8 vehicles entered each of the utilized spaces making an average of 1.5 vehicles that entered the expressway for each available 5-second space. The detailed data show that vehicles which utilized spaces less than 4 seconds long came almost entirely from the right-hand lane of the ramp, whereas approximately half of the vehicles that entered any space longer than 4 seconds came from the left lane of the ramp.

Lane usage

Information of the type shown by figure 45 in combination with lane usage on an expressway and the frequency of occurrence of various time spaces between vehicles at different hourly volumes may be used to determine the maximum number of vehicles that can enter an expressway from a ramp when there is no acceleration area. With uninterrupted flow, the frequency of occurrence of various time spaces between vehicles in a lane is very consistent for a given traffic volume regardless of the other prevailing conditions, and can be predicted with a high degree of accuracy from the results of comprehensive studies (see figs. 9 and 10 in part IV). Lane usage, or the percentage of vehicles in each lane, will vary, however, for different total traffic volumes and for a large number of physical and geometric conditions that might prevail in the vicinity of a ramp.

On a modern four-lane express highway at tangent locations that are not in the vicinity of an intersection, traffic will generally be distributed between the two lanes for the one direction of travel approximately as shown by figure 46. **As the traffic approaches an interchange or a ramp, the distribution between lanes will change and the extent of the change will depend on the particular lay-out at the interchange, the amount of traffic that enters and leaves the expressway at the interchange or ramp, and the effectiveness of the directional signs and pavement markings.**

Figure 47 shows the distribution of traffic between the lanes on an expressway at the approach to a ramp on the right with a continuous back-log of vehicles trying to enter the expressway. In the one case, as shown by the solid line, traffic from the ramp was required to stop before entering the expressway whereas, in the other case, as shown by the broken line, an acceleration lane was available and entering traffic did not necessarily stop.

For both conditions shown in figure 47, most of the traffic on the expressway used the second or left-hand lane during low traffic volumes to avoid interference with traffic entering from the ramps. **As the traffic volume on the expressway increased, however, the distribution between lanes became more nearly equal, there being a somewhat greater tendency for the drivers to avoid the right-hand lane when entering traffic was not required to stop than when it was required to stop. With an acceleration area available, therefore, it is not only easier to perform the merging operation than when there is a stop sign on the ramp and no acceleration area, but there are more spaces between vehicles in the first lane of the expressway that can be utilized by traffic entering from the ramp.**

The most important point illustrated by figure 47, however, is that **the effect which the ramp has on the distribution of traffic between lanes during low traffic volumes on the expressway is not apparent when the volume exceeds 1,000 vehicles per lane. Drivers will not crowd one lane to leave another lane comparatively free of traffic so that other drivers from a ramp can enter more**

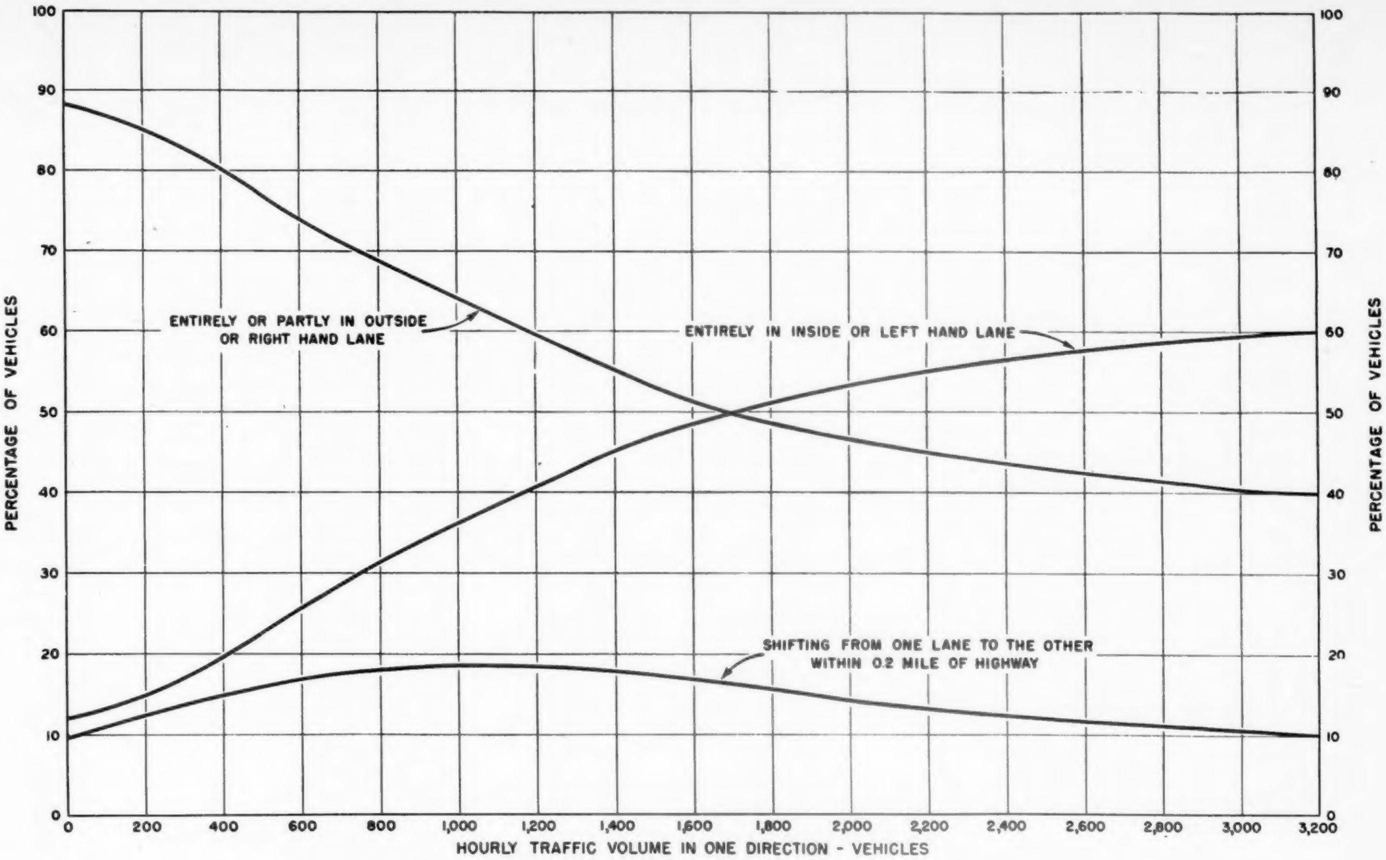


Figure 46.—Distribution of vehicles between traffic lanes on a four-lane highway during various hourly traffic volumes.

easily. This is one reason that it is not proper to assume that as many vehicles can enter an expressway from a ramp as the difference between the capacity of the expressway beyond the ramp and the traffic volume on the expressway ahead of the ramp.

The possible capacity of a particular ramp in relation to the volume of traffic on an expressway is shown by figure 48. The design at this location provided a stop sign to control traffic from the ramp. Recently, however, police-officer control has been necessary during peak periods to handle the increased traffic volumes without long delays to traffic on the ramp. Stopping through traffic on a facility designed for freeway or expressway operation is extremely undesirable but, as in this case, such a procedure will eventually have to be resorted to on many of the freeway facilities now being designed unless careful consideration is given to capacity aspects of the designs employed in the vicinity of ramps. In at least one case, it has been necessary to prohibit the use of a ramp during peak periods to prevent traffic on the entire expressway from coming to a standstill.

The conditions illustrated by figure 48 represent an expressway with a maximum possible capacity of 3,050 vehicles per hour in the one direction and a ramp with a maximum possible capacity of 2,480 vehicles per hour. Neither of these capacities could be realized, however, except when the traffic volume on one of the two was zero. The greatest difference in total capacity between police-officer and stop-sign control occurred when the volume approaching on the expressway was about 1,500 vehicles per hour. Under this condition, officer control increased the total flow of vehicles from 2,130 per hour to 2,900 per hour, or 36 percent. Even with this increase, however, the traffic that could enter from the ramp was limited to 1,400 vehicles

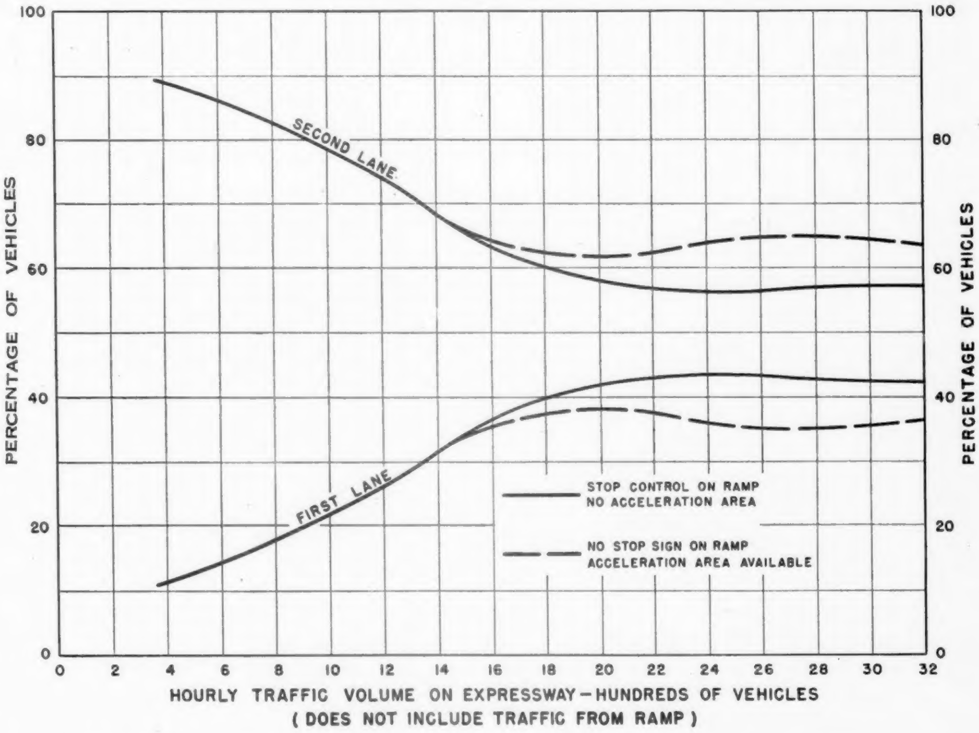
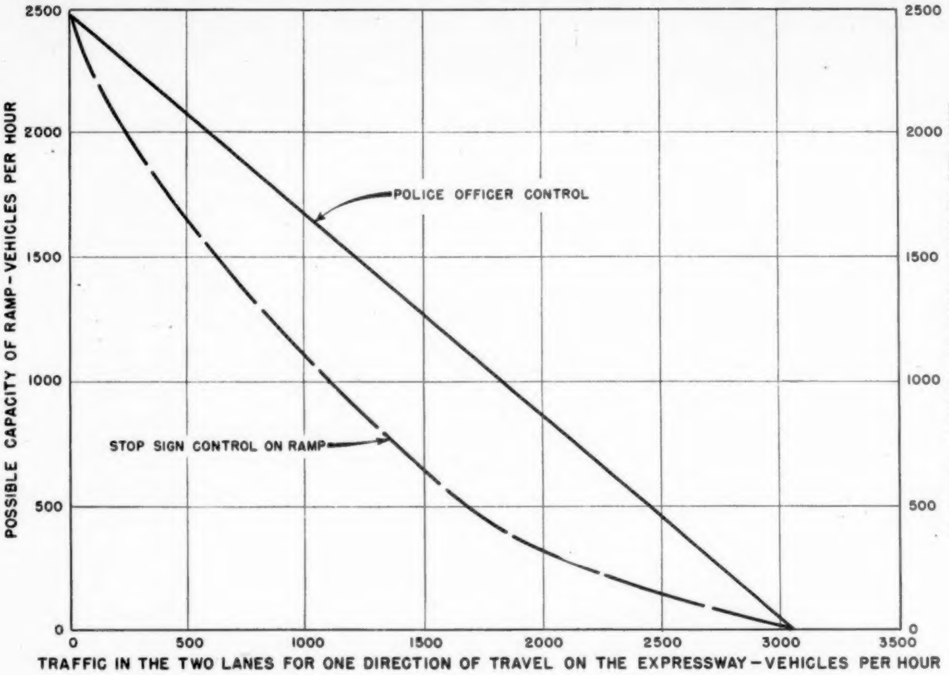


Figure 47.—Distribution of vehicles between traffic lanes on a four-lane expressway at approach to ramp where heavy volume enters the expressway on the right.



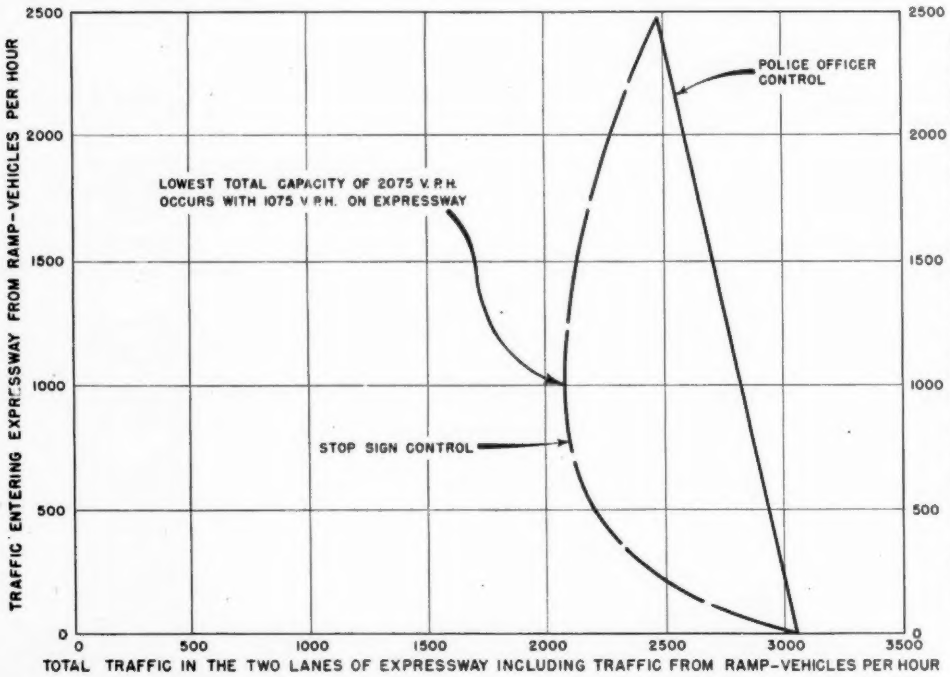


Figure 48—Variation in ramp capacity with volume of traffic on an expressway: ramp 20 feet wide, with no acceleration area.

per hour, or only 90 percent of the volume needed to fill the expressway to its capacity.

Exit from expressway to ramp

In addition to the actual geometric features near the point where traffic leaves an expressway to enter a ramp, the maximum volume of traffic that can enter a ramp without causing unsatisfactory operating conditions on the expressway is influenced (1) by the total volume of traffic on the expressway, (2) by the percentage of this total volume that desires to use the ramp, and (3) by the percentage of commercial vehicles. Regardless of the general lay-out, within reasonable limits, 1,200 to 1,500 vehicles per hour can enter a ramp in one line if all of these vehicles and no other vehicles are in the lane adjacent to the ramp. Likewise, two lines of traffic or 2,400 per hour can enter a ramp if all of these vehicles are in the two lanes of the expressway adjacent to the ramp and the drivers in the second line can be sure that all vehicles in the first line are headed for the ramp.

A perfect segregation of the traffic bound for a ramp seldom occurs ahead of the ramp, especially when an expressway is loaded to capacity or near capacity volumes. Some through vehicles, especially those driven by the slow, cautious drivers or the drivers looking for destination signs, and the commercial vehicles, will be in the right-hand lane of the expressway. If a ramp is on the right, these vehicles occupy spaces in the lane that could otherwise be used by vehicles entering the ramp. They also cross the paths of vehicles entering the ramp from the second lane, thereby creating a hazardous condition whenever the traffic volume entering the ramp exceeds 1,200 to 1,500 vehicles per hour, which require the use of two lanes.

The tendency of some drivers of through vehicles to stay in the right-hand lane where other traffic leaves the expressway by a ramp on the right is illustrated by figure 49. The distribution by lanes at the low traffic volumes differs greatly from the distribution found on sections where there are no ramps (fig. 46). It also differs from the distribution at the approach to a ramp used by traffic entering an expressway (fig. 47). At the near-capacity volumes, however, the difference for the various conditions is much less marked, there being a tendency in all cases for traffic to be fairly evenly divided between the lanes.

CONCLUSIONS

Although the data presented in the foregoing discussion show ramp capacities for only a few specific conditions, application of the information relative to lane usage on expressways is of assistance in rationalizing the problems concerning ramp capacities under a wide variety of conditions.

The following conclusions briefly summarize the more important facts concerning ramp capacities:

1. The capacity of a ramp may be limited by either the width and alinement of the ramp, or by traffic and physical conditions at either terminal.

2. With the curvatures and traffic conditions that are generally present at ramp locations, it is usually difficult to obtain better than the equivalent of single-lane operation on the ramp. With such operation, the ramp capacity normally will not exceed 1,200 passenger cars per hour with a possible maximum of 1,500 passenger cars per hour when vehicles can keep moving at a speed of 15 to 20 miles per hour (minimum radius about 100 feet). Serious consideration should be given to the provision of additional ramps when the traffic demand exceeds 1,200 vehicles per hour on any one ramp, unless conditions required for safe two-lane operation can be satisfied. Conditions that must be present to enable a ramp to accommodate 1,200 vehicles per hour are:

(a) Adequate ramp width for off-tracking and turning requirements of commercial vehicles (note: The 1,200 figure is for passenger cars and must be lowered to compensate the effect of commercial vehicles that are present. As a minimum, each commercial vehicle with dual tires has the effect of two passenger cars).

(b) Adequate surface or shoulder width for disabled vehicles, especially on up ramps.

(c) Adequate speed change areas, or excess capacity of the roadways connected by the ramp.

3. It is possible for a ramp and its terminals to handle traffic volumes exceeding 1,200 vehicles per hour under certain conditions. The following are some of the conditions that must be satisfied:

(a) Adequate surface width and radius of curvature (at least a 28-foot surface and a minimum radius of about 200 feet).

(b) At least one through lane on each highway entirely free of traffic other than that using the ramp, and a second lane on each highway sufficiently free of through traffic to accommodate the ramp volume which is in excess of 1,200 vehicles per hour (generally the exit and entrance to the ramp must have design characteristics similar to those at a Y intersection of two highways).

4. In considering the availability of spaces in the through traffic lanes into which vehicles from the ramp may enter, the following factors are important:

(a) At traffic volumes below 1,000 vehicles per lane per hour on a multilane highway, there is a marked tendency for traffic approaching an entering ramp to shift away from the lane adjoining the terminal of the ramp, thereby avoiding interference with vehicles from the ramp and providing a better opportunity for vehicles to enter the highway. When the volume in one direction on the highway exceeds 1,000 vehicles per hour per lane, however, this tendency to shift away from the lane adjoining the ramp does not occur, and traffic maintains approximately the same distribution between the lanes at the ramp terminal as on sections where there are no ramps. This is particularly true at locations where there are stop signs for vehicles entering from the ramp. Where there are acceleration areas without stop-sign control for vehicles from the ramp, a somewhat higher percentage of the through traffic will use the lane farthest from the ramp than normally uses this lane at locations removed from the influence of ramps.

(b) Because of the tendency of traffic to distribute itself in all lanes of a multilane highway during heavy traffic volumes, thus avoiding a more crowded condition in one lane than in another lane, many of the openings between vehicles in the lane or lanes that are not adjacent to the ramp terminal are shielded against occupancy by vehicles entering from the ramp. As a practical matter, it is impossible to fill all the available openings between cars on an expressway unless there are suitable acceleration areas where merging can be performed. Hence the volume of traffic that can enter from a ramp will seldom equal the amount by which the capacity of the expressway exceeds the volume of traffic on the expressway ahead of the ramp.

5. Where points of access and egress to and from the expressway are closely spaced, the resulting interchange of traffic between lanes may cause certain lengths of the expressway to fall within the category of weaving sections and in that way affect the capacity of the expressway and the ramps.

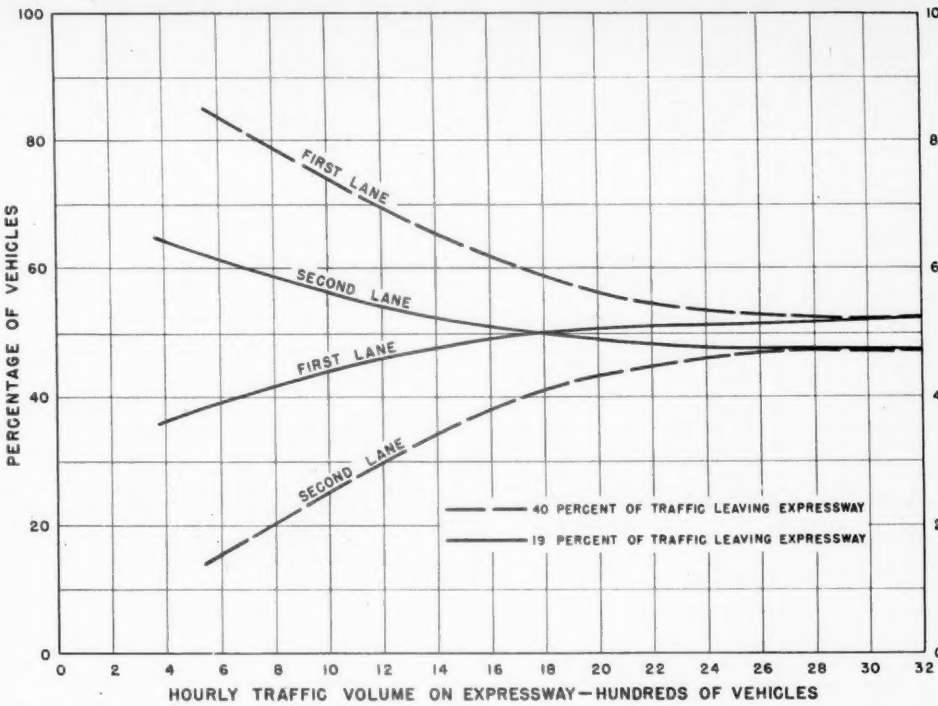


Figure 49.—Distribution of vehicles between traffic lanes on a four-lane expressway at approach to ramp where heavy volume leaves the expressway to the right (hourly traffic volume on expressway includes traffic leaving expressway at the ramp).

6. In selecting the hour of peak traffic movement, a knowledge of the distribution of traffic between the ramp and the expressway is of utmost importance. An hourly pattern for traffic using a ramp is of little worth without a similar pattern for the expressway or other through street.

7. For exit ramps, the number of vehicles that can enter the ramp is affected by the volume of through traffic using the right lane. Many slow-moving passenger cars and most commercial vehicles use this lane, thereby occupying spaces in the traffic stream that otherwise might be utilized by vehicles desiring to leave the expressway. For most installations, the volume of traffic using an exit ramp cannot exceed 1,200 passenger cars less the number of through vehicles occupying the lane adjacent to the ramp. Proper use of signs will aid greatly in minimizing the number of vehicles that must be deducted from the ramp capacity for this reason.

If an exit ramp is to accommodate more than 1,200 passenger cars an hour, the conditions enumerated under item 3 above must be satisfied.

Exit turn from an expressway at a grade separation. Two lines of traffic can enter a ramp if they are in the two lanes of the expressway adjacent to the ramp, and if the drivers in the second line can be sure that all vehicles in the first line are destined for the ramp.

Part VIII.—Relating Hourly Capacities to Annual Average Volumes and Peak Flows

Introduction

Throughout the foregoing discussion of highway capacities, traffic volumes have been expressed in terms of vehicles per hour. This time period was selected by the Committee because periods of congestion or heavy traffic movement usually are of relatively short duration, occupying only a small portion of the day. **An hour is the shortest period of time that conforms to current traffic counting practices although it is realized that, under certain conditions, counts for even shorter periods might be desirable.**

When trying to apply the hourly capacities to the solution of practical problems, the engineer often finds that complete and detailed hourly volume data are not available for the facility under consideration. Sometimes the only traffic information at hand is the annual average 24-hour volume, based either on a few counts scheduled throughout the year or estimated from counts made at nearby locations. In other instances, the only traffic volume information available may be a count for a single day, counts for a few scattered days during the year, or, as oftentimes occurs, counts for periods shorter than 24 hours. **Although the cost of obtaining comprehensive traffic data is relatively inexpensive, in comparison with the cost of any extensive improvement, it is oftentimes necessary to make use of very meager data in the solution of problems involving highway capacity. In such cases, a method for adjusting the available counts to obtain the needed hourly capacities becomes a matter of paramount importance, and a clear understanding of the variations which may be expected in the traffic load is necessary.**

Without this knowledge, the application of traffic-count data to the planning and design of new facilities or to the proper operation of existing facilities cannot be completely successful. It is the purpose of this chapter to supply information regarding these questions, which are so directly related to highway capacity and necessary for the proper use of the capacity data.

Relation of Annual Average Volume to Capacity

The most common measure used in reporting the traffic on a highway is the annual average daily traffic volume. Although such a figure is of value for many purposes, such as determining the type of surface that should be provided and in computing the total earnings derived from gasoline taxes by travel on the highway, it is not a direct measure of the number of needed traffic lanes or of other geometric features of the highway necessary to serve adequately the drivers using the highway.

Proper use of signs plays an important part in preventing or alleviating congestion.

Days during which the actual volume corresponds to the average day of the year occur much less frequently than is commonly supposed. A study of 48 selected rural highway locations in 45 States shows that the traffic volume representing the annual average daily volume is exceeded during 160 days of the 365 days in a year at the average location. At some locations, the average daily volume was exceeded on as few as 70 days a year and at other locations it was exceeded on as many as 228 days during a year.

It is therefore evident that facilities planned to accommodate only the traffic of an average day will be definitely overtaxed for a considerable portion of the year. In fact, at the average rural location the volume on certain days will be more than double the annual average. The practice of expressing capacities of various facilities in terms of average daily traffic without considering the variation in traffic flow and the relation between the average daily volumes and the peak hourly volumes has contributed in a large measure to the misunderstanding and lack of agreement regarding highway capacities.

Selection of Required Hourly Capacities From Continuous Counts

Closely related to the fluctuation in traffic flow during the various hours of the day, days of the week, and seasons of the year, is the selection of the specific hourly volume that should be used for design purposes or as the reasonable volume which an existing street or highway facility should be expected to accommodate. It is for these purposes that the time period of 1 hour assumes primary importance.

If a roadway facility is to be so designed that traffic will be properly served, consideration must be given to the brief but frequently repeated rush-hour periods. It is neither wise nor economical, however, to provide for the extreme hourly volumes of traffic that may occur but a few times during a year. The law of diminishing returns must be applied to fix the highest hourly volume which will justify the necessary expenditure of funds to provide the added capacity.

When hourly traffic counts for a full year are available for a highway under consideration, it is possible to show the yearly traffic pattern by arranging the hourly volumes in descending order of their magnitude. Such a table or graph greatly simplifies the problem of selecting the minimum hourly capacity which will adequately serve present traffic for the particular highway, or a future traffic demand as based on an assumed or estimated percentage increase.

An example of how yearly traffic patterns can be used to select the minimum hourly capacity which will adequately serve the traffic presently using the highway is presented in table 21. This table shows the yearly traffic patterns for two rural highway sections in the same State, both having an annual average volume of 3,000 vehicles per day. Data for Sunday and holiday traffic are included in the table because **there is no sound basis for any assumption that there is less need to provide adequate capacity for peak hours on Sundays and holidays than on weekdays.**

Although both roads have the same annual traffic volume, the peak hourly traffic volumes on road *B* are much higher than those on road *A* because of the greater seasonal and weekly fluctuation in traffic flow. On road *A*, 1 percent of the vehicles use the road when the hourly traffic volume is 500 vehicles or more. On road *B*, however, more than 20 percent of the vehicles, or about 200,000 per year, are traveling when the hourly traffic volume is

Table 21.—*Yearly traffic patterns for two rural highway sections on each of which the average annual daily traffic is 3,000 vehicles*

Hourly traffic volume	Percentage that hourly volume is of average 24-hour volume	Road A		Road B	
		Number of hours during year when hourly volume was exceeded	Percentage of total vehicles using road when hourly volume was exceeded	Number of hours during year when hourly volume was exceeded	Percentage of total vehicles using road when hourly volume was exceeded
3,000	100.0	0	0	0	0
1,300	43.3	0	0	0	0
1,200	40.0	0	0	6	0.69
1,100	36.7	0	0	15	1.64
1,000	33.3	0	0	29	2.99
900	30.0	0	0	62	5.88
800	26.6	0	0	100	8.50
700	23.3	1	0.06	153	12.43
600	20.0	5	.30	206	15.63
500	16.7	20	1.01	298	20.24
400	13.3	111	4.60	430	25.67
300	10.0	360	12.29	727	35.15
200	6.7	1,366	34.04	1,388	50.05
100	3.3	5,270	84.21	3,510	78.43
0	0	8,760	100.00	8,760	100.00

500 vehicles or more; and over 8 percent, or about 80,000 per year, are traveling when the hourly traffic volume is 800 or more vehicles.

Thus, if the practical capacity of road *A* is only 450 vehicles per hour it will be no more congested than will road *B* if the practical capacity of the latter road is 1,000 vehicles per hour. In both cases the practical capacity would be exceeded during about 30 hours of the year and about 3 percent of the total traffic using each road during the year would be inconvenienced by congestion during the 30 hours.

The yearly traffic patterns also show that relatively few drivers would be benefited by roadways of higher practical capacities than these, and an increasingly large percentage of drivers would be inconvenienced by roadways with only slightly lower practical capacities.

Ordinarily the foregoing procedure can be followed in the selection of a practical hourly capacity for an estimated future increase in traffic by merely changing the hourly traffic volumes shown in the first column of table 21 by the same percentage as the estimated increase in the annual volume. **Hourly volumes increase in about the same ratio as do average daily volumes on any roadway.**

Determining Required Hourly Capacity from Annual Volume

The preceding discussion and table 21 illustrate the wide difference that might exist in the peak hourly traffic flows on two highways having the same annual traffic volumes, and the resulting difference in hourly capacities of two highways necessary to provide approximately the same traffic service under the two conditions.

As an aid in relating the annual traffic volumes to the peak hourly flows, the results of an analysis of traffic count data for 171 stations in 48 States are summarized in table 22 and detailed in table 23. It is anticipated that these tables will be of value in estimating peak hourly flows on roads within the area of influence of these stations, or where similar conditions exist, when only the annual traffic volumes are known.

Figure 50 shows the average yearly traffic pattern for these 171 rural highway locations and an indication of the range in peak flows that occur at different locations. On a road

Table 22.—*Variations in traffic flow on main rural highways averaged for the various census regions (prewar data)*

Census region	Locations studied	24-hour volumes			Percentage of average 24-hour volume in certain hourly volumes during year				
		Average for year	Percentage of average in—						
			Maximum 24 hours	Tenth highest 24 hours	Maximum hour	Tenth highest hour	Twentieth highest hour	Thirtieth highest hour	Fiftieth highest hour
		Vehicles	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
New England	17	3,657	316.9	214.9	34.5	25.6	23.1	21.7	19.6
Middle Atlantic	10	13,884	241.4	190.2	21.9	17.2	16.0	15.2	14.2
South Atlantic:									
North portion	11	4,349	207.2	166.4	20.1	16.6	15.5	14.8	13.8
South portion	12	2,595	161.8	135.1	18.3	14.3	13.3	12.7	11.9
East North Central	22	2,864	263.7	200.0	29.0	19.6	17.8	16.6	15.3
West North Central	20	2,017	250.6	178.5	29.4	18.9	16.7	15.7	14.4
East South Central	12	2,420	187.2	151.9	23.7	17.6	16.3	15.6	14.3
West South Central	14	3,108	177.6	140.8	17.7	13.9	12.9	12.3	11.5
Mountain	36	1,940	216.1	165.9	23.7	16.0	14.6	13.9	12.8
Pacific	[1] 13	2,975	220.5	167.4	24.1	16.7	15.0	14.2	13.1
Total	167	3,370	228.9	172.6	24.9	17.8	16.2	15.3	14.1

[1] Does not include four stations in California where traffic is highly seasonal and not representative of most roads in the State.

Table 23.—*Variations in traffic flow on main rural highways*

Region and State	Recorder station No.	Route No.	Year starting	24-hour volumes			Percentage of average 24-hour volume in certain hourly volumes during year				
				Average for year	Percentage of average in—						
					Maximum 24 hours	10th highest 24 hours	Maximum hour	10th highest hour	20th highest hour	30th highest hour	50th highest hour
NEW ENGLAND:				*Vehicles*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
Connecticut	6-WB	[1] 1	Mar. 31, 1939	6,813	287.7	244.3	29.7	25.6	24.0	23.4	22.0
	7-EB	1	do	6,811	297.2	221.4	26.7	21.7	19.6	18.5	17.5
	6 and 7	1	do	13,624	292.5	227.9	25.7	21.2	19.2	18.6	17.7
	17	5	do	8,313	155.3	136.8	13.3	12.0	11.5	11.2	10.8
Maine	3	27	Dec. 15, 1940	1,397	319.2	240.9	39.8	28.6	25.8	24.3	22.0
	2	1	Feb. 5, 1938	1,287	274.1	263.4	22.4	19.8	18.8	17.9	17.1
	2	1	Jan. 1, 1940	1,248	274.3	231.6	24.0	20.6	19.5	18.7	17.6
	4	17	do	436	354.8	183.9	39.4	21.6	19.5	18.1	16.7
Massachusetts	8	1	Apr. 30, 1938	7,363	288.2	246.9	24.6	22.2	20.7	19.9	18.2
	10	10	July 21, 1939	6,476	218.7	189.0	18.8	16.8	16.3	15.8	15.0
	1	8	Jan. 1, 1940	3,071	205.4	160.6	29.3	20.7	17.4	16.3	14.6
New Hampshire	2	3	May 12, 1940	6,635	263.3	198.8	28.5	20.6	18.7	17.8	16.4
	1	3	Jan. 1, 1940	1,410	517.2	277.8	55.2	43.0	37.5	32.8	27.5
	1	3	Sept. 18, 1937	1,360	647.6	279.9	60.4	48.4	38.7	34.1	28.9
	3	[2] 104	Oct. 22, 1939	535	298.1	227.1	34.7	25.4	23.7	22.8	20.9
Rhode Island	4	[2] 146	Jan. 1, 1940	4,609	197.2	166.9	26.3	21.3	19.6	18.5	16.6
	2	1	do	2,155	452.9	263.9	57.2	41.6	37.1	34.0	30.0
	2	1	June 4, 1938	1,931	466.8	238.0	68.3	43.8	37.9	34.2	31.4
	3	[2] 102	Jan. 1, 1940	330	240.9	215.2	62.7	27.6	26.1	24.8	22.7

See footnotes at end of table, p. 271.

Table 23.—*Variations in traffic flow on main rural highways*—Continued

Region and State	Recorder station No.	Route No.	Year starting	24-hour volumes: Average for year	24-hour volumes: Percentage of average in—Maximum 24 hours	24-hour volumes: Percentage of average in—10th highest 24 hours	Percentage of average 24-hour volume in certain hourly volumes during year: Maximum hour	10th highest hour	20th highest hour	30th highest hour	50th highest hour
				Vehicles	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
New England—Continued											
Vermont	A-12-2	2	Jan. 1, 1949	1,688	236.3	196.1	38.1	19.5	17.5	16.9	16.2
	A-12-2	2	Nov. 28, 1936	1,615	276.5	219.6	31.3	22.2	20.9	20.3	18.0
	C-14-2	[2] 14	Jan. 1, 1941	1,145	275.1	194.8	34.1	23.3	20.5	19.0	17.6
	B-6-1	7	Mar. 2, 1941	352	706.0	280.1	69.9	44.5	40.0	37.2	30.3
Middle Atlantic:											
New Jersey (Essex County)	NB	25	Jan. 1, 1941	32,100	144.2	130.6	14.2	11.9	10.9	10.5	9.2
	SB	25	do	30,150	159.9	137.8	13.3	11.4	11.1	10.9	10.3
	NB and SB	25	do	62,250	143.6	131.6	11.2	9.6	9.3	9.3	8.8
New Jersey (Edison Bridge)	NB	35	do	13,725	257.9	167.3	26.7	23.3	21.1	19.7	18.0
	SB	35	do	11,656	436.3	315.3	36.0	28.0	24.7	24.7	23.6
	NB and SB	35	do	25,381	284.3	231.6	24.0	19.1	17.3	16.2	15.4
New Jersey (Woodbridge)	NB	35	do	11,286	229.3	181.4	30.1	20.7	18.4	15.9	8.9
	SB	35	do	10,766	489.0	255.3	41.5	30.1	26.2	21.7	18.9
	NB and SB	35	do	22,052	299.2	211.5	26.8	18.0	17.0	15.9	13.6
New York	8-6	17	Feb. 25, 1940	7,594	291.7	255.1	21.1	19.6	18.8	18.3	17.6
	2-1	[2] 5	Jan. 1, 1940	4,794	156.4	138.7	15.4	11.7	11.3	11.1	10.5
	5-1	[2] 5	Dec. 31, 1938	4,458	329.2	211.5	27.2	22.7	19.9	18.6	17.4
	5-3	33	Jan. 1, 1939	2,843	186.1	165.5	20.7	17.9	16.9	16.0	14.4
Pennsylvania	1	20	Jan. 1, 1941	6,112	291.3	212.7	23.9	20.0	17.2	16.2	15.3
	1	20	Nov. 20, 1937	4,395	311.1	209.6	28.1	19.3	17.2	16.0	14.9
	17	[2] 53	Jan. 1, 1940	2,979	160.4	141.6	18.7	13.7	13.1	12.7	12.1
	4	6	July 24, 1937	1,231	261.9	203.7	27.8	20.1	20.1	18.3	17.1
South Atlantic (North portion):											
Delaware	C-NB	13	June 8, 1941	3,371	344.9	235.9	40.1	34.6	31.4	29.2	25.6
	C-SB	13	do	3,370	287.5	202.1	25.4	21.1	17.7	16.9	15.3
	C-Comb	13	do	6,741	254.2	206.5	23.8	20.5	20.0	19.1	16.9
	E	13	May 1, 1941	3,400	201.3	158.4	17.7	14.4	12.9	12.1	11.3
Maryland	12	40	Jan. 22, 1938	7,250	227.0	171.1	22.3	18.2	16.6	15.5	14.3
	11	1	Jan. 1, 1940	5,457	215.5	171.6	27.3	20.4	18.8	17.6	16.2
	2	40	Apr. 3, 1937	3,030	225.5	190.3	23.5	16.8	16.1	15.4	14.7
Virginia	1	1	June 26, 1937	6,668	216.3	174.5	20.4	16.9	15.6	14.9	13.9
	2A	11	May 5, 1940	2,600	214.2	163.7	17.3	15.9	15.0	14.3	13.5
	4A	58	Jan. 31, 1939	2,490	151.2	142.7	13.6	12.6	11.9	11.6	11.0
West Virginia	3	60	Dec. 8, 1939	7,285	170.5	131.0	12.5	11.2	10.6	10.5	10.2
	1	[2] 10	June 4, 1939	1,727	169.2	141.9	17.6	15.7	14.9	14.4	13.7
	11	[2] 2	Jan. 1, 1940	1,258	234.7	178.3	25.5	20.2	17.6	17.1	15.7
South Atlantic (South portion):											
Florida	4	90	Jan. 1, 1941	3,646	147.3	132.1	13.3	11.9	11.3	10.9	10.3
	4	90	May 15, 1937	3,365	144.2	119.7	14.9	10.8	10.3	9.8	9.2
	3	41	Jan. 1, 1938	1,668	147.3	129.0	13.8	11.6	11.2	10.8	10.4
	1	90	Nov. 27, 1937	749	144.9	125.6	24.4	13.0	12.3	11.2	10.3
Georgia	3	29	Jan. 1, 1940	4,307	153.5	140.4	16.7	15.6	14.7	14.1	13.3
	1	41	Jan. 1, 1939	3,238	142.9	126.7	14.2	12.7	12.2	11.5	11.1
	12	84	do	632	170.6	135.9	20.9	15.0	13.8	12.2	11.4
North Carolina	3	29	do	4,296	174.4	135.5	25.0	16.8	14.5	13.9	12.6
	4	19	Feb. 25, 1939	2,540	163.2	144.2	16.4	15.2	13.9	13.2	12.4
	12	158	Jan. 1, 1941	1,414	218.8	157.6	20.6	17.1	15.9	15.4	14.6
South Carolina	5	29	Feb. 20, 1937	3,936	154.5	133.4	15.9	15.0	14.5	13.9	13.0
	7	13	Jan. 1, 1940	3,268	174.7	137.2	20.4	14.7	14.0	13.4	12.9
	2	15	Dec. 4, 1937	1,583	151.3	129.6	16.9	13.4	12.4	11.9	11.2
East North Central:											
Illinois	1	45	Sept. 27, 1936	4,057	443.7	369.7	31.8	29.6	28.1	27.1	25.9
	2	66	Jan. 24, 1937	3,937	233.2	185.3	25.0	18.3	16.4	15.1	13.8
	7	50	Dec. 18, 1937	3,210	171.9	126.4	15.9	14.1	13.1	12.5	11.3
Indiana	2A	20	Aug. 28, 1937	3,490	226.8	186.5	25.8	18.3	17.2	16.1	13.7
	34A	30	Dec. 21, 1940	3,198	224.2	186.2	17.6	15.6	14.2	13.4	12.3
	59A	40	Jan. 15, 1938	3,125	233.2	161.1	33.2	14.9	13.9	12.8	12.4
	42A	52	July 3, 1937	3,071	291.7	179.0	43.2	17.3	15.5	14.4	13.8
	72A	31	Jan. 15, 1938	2,293	213.8	165.9	20.4	15.3	13.4	13.0	12.1
Michigan	676	27	Oct. 2, 1937	3,151	235.3	200.6	53.9	20.7	17.2	16.3	15.3
	678	23	Jan. 1, 1941	1,538	336.0	246.8	36.4	28.3	26.0	24.3	23.0
	678	23	Jan. 1, 1939	1,200	300.9	267.5	44.8	31.2	26.2	24.8	22.5
Ohio	27	25	Feb. 18, 1939	3,928	213.7	156.9	23.1	15.3	13.8	13.3	12.5
	25	42	Apr. 12, 1939	3,645	214.4	158.5	21.2	14.7	13.6	13.1	12.4
	28	75	June 25, 1939	1,055	180.6	158.2	19.9	16.8	15.4	14.3	13.4
Wisconsin	2 and 3	[2] 41	Jan. 8, 1938	5,674	305.9	220.2	29.1	22.2	18.9	18.1	16.3
	2-WB	[2] 41	do	2,817	405.6	241.8	43.7	30.7	25.6	22.9	20.2
	3-EB	[2] 41	do	2,857	281.6	220.3	22.1	19.7	17.8	16.4	15.3
	3-EB	[2] 41	Jan. 1, 1941	3,921	439.8	217.0	30.4	23.4	20.5	18.9	16.0
	13	[2] 41	Oct. 4, 1939	4,870	263.6	234.8	26.0	22.4	21.1	19.9	18.1
	14	10	Jan. 1, 1940	2,953	248.4	211.1	30.0	18.5	17.2	15.8	14.6
	18	[2] 13	do	2,780	317.5	229.4	32.1	21.3	19.6	18.0	16.8
	5	[2] 41	do	2,199	203.6	163.8	31.4	17.1	15.1	13.2	12.4
	15	45	Jan. 6, 1939	2,037	289.8	211.1	24.5	20.1	18.4	16.7	15.5
	10	10	Jan. 9, 1937	1,632	364.2	242.3	28.5	19.9	19.4	18.0	17.0
	16	[2] 13	Jan. 1, 1940	1,044	294.1	189.1	24.3	19.6	18.0	16.3	14.4
	20	[3] P&S	do	282	312.4	206.7	40.8	29.4	26.2	23.4	20.2
West North Central:											
Iowa	601	65	Jan. 1, 1938	3,539	223.3	157.4	26.7	16.6	14.9	14.2	13.2
	601	65	Dec. 19, 1936	3,290	232.4	154.2	18.1	14.9	14.3	13.4	12.3
	69-616	69	Jan. 1, 1941	1,417	243.2	184.3	25.5	16.8	15.7	14.9	13.8
Kansas	5	24	Aug. 14, 1938	2,183	185.3	155.0	14.9	13.1	12.6	12.4	11.9
	3	50S	Feb. 18, 1939	2,059	164.9	146.2	17.1	14.3	13.3	12.2	11.5
	6	24	Jan. 1, 1940	991	162.9	147.7	18.7	12.9	12.0	11.7	11.2
Minnesota	157	212	Mar. 20, 1937	4,875	221.5	191.1	22.9	19.7	18.7	17.8	16.6
	159	10	Sept. 11, 1937	3,730	323.9	226.2	33.7	26.1	22.9	19.7	17.4
	175	52	July 3, 1937	872	264.8	183.0	28.9	22.2	18.2	17.4	15.0
Missouri	9	66	Jan. 23, 1939	5,220	339.3	257.4	28.4	25.1	22.1	21.4	20.3
	12	69	Jan. 1, 1939	3,307	200.2	165.4	28.9	16.1	14.8	14.3	13.7
	5	54	July 17, 1937	1,708	433.4	255.1	46.3	31.6	28.3	27.1	23.7
Nebraska	5	6	Jan. 8, 1938	2,128	238.9	166.8	48.8	28.0	17.0	14.9	13.9
	A4	77	June 15, 1940	2,096	326.9	139.1	27.4	14.6	11.4	9.9	9.2
	2	30	Jan. 8, 1938	1,619	200.9	178.9	16.9	13.8	13.3	12.9	12.6
North Dakota	111	10	Jan. 1, 1940	585	354.2	194.7	30.1	20.3	17.8	16.9	15.8
	102	[2] 1	Feb. 1, 1939	356	211.2	148.6	44.4	19.1	17.4	16.0	14.0
	103	2	Oct. 18, 1937	352	237.2	173.6	28.4	18.8	18.8	17.3	14.2
South Dakota	104A	77	Jan. 1, 1940	1,966	197.7	160.9	21.8	16.5	14.8	14.2	13.2
	101	14	May 15, 1937	982	290.8	199.1	54.4	18.2	16.8	16.3	15.7
	106	18	Dec. 31, 1938	479	186.0	140.3	27.6	14.4	13.4	12.9	11.9

See footnotes at end of table, p. 271.

Table 23.—*Variations in traffic flow on main rural highways*—Continued

Region and State	Recorder station No.	Route No.	Year starting	24-hour volumes: Average for year	24-hour volumes: Percentage of average in— Maximum 24 hours	24-hour volumes: Percentage of average in— 10th highest 24 hours	Percentage of average 24-hour volume in certain hourly volumes during year: Maximum hour	10th highest hour	20th highest hour	30th highest hour	50th highest hour
				Vehicles	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
EAST SOUTH CENTRAL:											
Alabama	5	11	Jan. 1, 1940	5,718	159.0	147.8	18.9	16.7	16.1	15.4	14.3
	4	78	Dec. 25, 1937	1,073	203.1	155.8	27.9	19.6	18.6	17.9	16.7
	2	72	Jan. 1, 1939	531	154.4	134.8	27.7	17.5	15.4	14.9	13.7
Kentucky	7	31W	Jan. 1, 1941	7,937	166.1	139.7	19.6	15.9	15.1	14.6	13.6
	9	41	Dec. 29, 1940	2,086	186.5	143.3	23.8	13.6	12.4	12.2	10.6
	12	23	May 4, 1941	1,323	166.7	141.5	16.3	14.6	13.5	12.6	11.7
Mississippi	3	51	Jan. 1, 1940	1,871	114.6	124.9	17.0	13.3	12.5	12.0	11.5
	8	45	do	1,671	177.4	128.1	22.9	16.6	15.1	14.0	12.7
	10	11	do	1,617	165.1	156.0	29.6	17.2	16.3	15.0	13.8
Tennessee	1	31W	Apr. 21, 1939	3,425	142.1	135.5	17.8	14.7	13.7	13.1	12.5
	3	[2] 71	July 14, 1939	990	410.4	274.3	44.1	36.5	31.9	30.6	27.8
	4	57	Feb. 9, 1941	794	170.8	141.3	19.1	15.4	14.9	14.4	13.0
WEST SOUTH CENTRAL:											
Arkansas	14	61	July 23, 1940	3,057	159.7	141.6	12.6	11.0	10.7	10.5	10.0
	13	70	Jan. 1, 1939	2,321	148.3	133.6	13.7	11.4	11.1	10.7	10.3
	17	[2] 22	Jan. 28, 1940	1,501	143.4	128.1	15.4	12.3	12.1	11.7	10.7
	11	63	Jan. 1, 1939	311	305.5	172.7	31.8	23.5	19.9	18.0	15.4
Louisiana	4	90	Apr. 24, 1937	4,226	143.1	125.6	15.4	11.7	11.1	10.6	10.0
	1	79	Dec. 25, 1937	3,304	158.6	124.5	16.0	13.4	12.0	11.6	11.0
	14	90	Jan. 1, 1940	3,159	139.6	127.5	14.8	12.0	11.4	11.0	10.6
Oklahoma	6	66	Jan. 1, 1939	4,291	170.9	126.9	16.0	12.3	11.9	11.2	10.5
	5	77	Feb. 27, 1937	2,259	259.0	180.3	19.0	16.7	15.7	15.3	14.0
	1	66	May 15, 1937	2,111	196.0	158.1	16.6	14.8	13.0	12.5	11.8
Texas	1	80	July 7, 1939	9,053	154.1	133.3	17.2	13.4	12.4	12.0	11.5
	4	77	Jan. 1, 1941	5,180	181.2	145.9	19.4	14.6	13.3	12.9	12.0
	4	77	Jan. 1, 1938	4,049	151.9	143.5	16.6	14.2	13.3	12.8	12.0
	5	80	Dec. 19, 1936	2,427	163.6	140.0	16.6	13.9	12.9	12.3	11.5
	8	81	Mar. 20, 1937	875	177.9	135.0	24.3	14.3	12.9	12.3	11.3
MOUNTAIN:											
Arizona	1	80	Jan. 1, 1941	8,757	157.4	130.1	16.3	12.2	11.7	11.2	10.5
	1	80	July 7, 1939	7,174	172.1	131.2	22.9	13.0	11.9	11.5	10.9
	4	60	Jan. 28, 1939	1,743	177.1	140.4	15.8	12.4	11.2	10.7	10.0
	2	80	Jan. 1, 1940	1,036	177.9	133.5	14.1	11.9	11.3	10.9	10.4
Colorado	11	85	June 26, 1938	5,472	138.3	129.7	14.1	12.4	11.7	11.1	10.5
	3	85	Feb. 27, 1937	4,334	214.6	173.7	21.9	16.0	15.0	14.3	13.4
	14	160	Jan. 1, 1940	1,286	327.9	151.2	46.0	18.4	14.7	14.5	12.4
	14	160	Jan. 1, 1941	1,224	319.9	160.0	46.1	17.4	15.9	13.6	12.4
	7	34	do	731	203.8	173.2	21.6	15.3	14.6	14.2	13.4
Idaho	2	30	Apr. 3, 1937	3,085	179.2	149.5	15.8	13.9	13.1	12.8	12.1
	1	10	Jan. 1, 1938	2,438	460.8	222.8	36.6	28.9	23.4	21.0	19.4
	3	30	do	2,290	165.9	139.0	17.0	12.2	11.5	11.3	10.6
Montana	A11	10	Dec. 3, 1939	2,777	213.4	165.0	18.4	13.7	12.9	12.4	11.6
	4	10	Oct. 29, 1938	982	232.8	177.1	19.9	17.4	15.0	14.5	13.5
	A7	91	June 30, 1939	495	280.2	196.2	36.0	22.2	18.8	18.0	16.2
Nevada	110	93	Jan. 1, 1940	1,752	239.1	139.3	19.1	16.2	13.8	11.4	10.7
	101	50	Nov. 6, 1937	1,469	165.2	130.6	20.5	15.7	12.8	12.3	11.0
	107	40	June 5, 1937	755	207.9	180.8	18.3	14.4	13.6	13.1	12.2
New Mexico	2	85	Jan. 1, 1941	3,375	173.7	146.2	15.2	13.7	13.2	12.8	12.4
	10	285	do	2,185	167.0	130.9	15.1	11.4	11.2	10.6	10.3
	8	[2] 28	do	1,804	165.2	140.2	18.6	12.5	11.9	11.6	11.3
	6	66	Jan. 15, 1938	1,574	194.5	160.0	18.9	15.1	14.6	13.7	12.7
	7	70	Aug. 7, 1937	1,461	179.6	154.8	20.7	11.8	11.4	11.1	10.3
	3	64	Jan. 1, 1941	1,456	191.7	167.6	17.7	13.7	12.7	12.5	11.9
	4	54	do	1,391	207.7	166.1	22.4	13.9	13.2	12.9	11.9
	11	70	do	1,324	204.7	152.5	19.5	13.6	12.8	12.3	11.6
	1	85	June 12, 1937	1,216	238.3	184.8	23.0	17.2	16.0	14.9	13.7
	1	85	Jan. 1, 1941	1,059	192.8	168.1	28.0	14.8	13.9	13.2	12.7
	5	[2] 18	do	1,163	148.8	126.8	23.9	13.7	12.9	12.4	11.7
	9	54	do	946	239.6	187.9	28.9	19.2	18.0	16.6	15.8
	9	54	Jan. 8, 1938	751	241.5	165.6	30.5	20.4	17.0	16.4	14.8
Utah	302	50	Jan. 1, 1941	4,422	231.5	164.4	17.7	15.4	14.5	14.2	13.4
	302	50	July 10, 1937	2,443	230.9	172.4	20.9	16.9	15.1	14.4	13.7
	301	40	Jan. 1, 1941	2,111	355.0	247.7	29.2	26.4	23.1	21.9	19.6
	301	40	Nov. 13, 1937	1,766	301.1	260.0	41.3	29.0	26.2	25.4	22.8
	305	89	Jan. 1, 1941	833	208.8	164.3	31.0	15.8	14.2	13.6	12.5
	308	40	do	557	234.3	210.4	27.8	19.7	18.9	17.7	16.7
Wyoming	204	30	do	1,647	233.3	198.7	31.8	17.7	16.6	15.8	14.3
	204	30	Jan. 1, 1939	1,257	234.4	197.9	26.6	18.8	17.1	16.5	15.0
	203	87	Jan. 1, 1940	1,551	180.3	161.3	29.8	15.4	15.0	14.5	13.0
	205	20	Jan. 1, 1941	1,375	216.7	152.7	20.1	13.3	11.8	11.0	9.5
	205	20	May 19, 1939	1,309	230.2	149.7	54.7	15.0	13.7	13.1	12.3
	206	30	Jan. 1, 1941	1,365	220.7	173.4	19.8	16.7	15.1	13.8	12.1
	207	14–16	do	555	286.5	253.5	30.8	24.7	22.1	20.2	16.0
PACIFIC:											
California	1	99	do	7,692	207.0	149.3	17.1	13.7	12.8	12.2	11.0
	1	99	July 10, 1937	5,815	197.2	159.9	18.9	15.3	13.5	12.4	10.8
	9 [4]	[2] 43	Jan. 1, 1941	5,228	254.0	236.0	27.6	24.7	23.1	22.4	----------
	2	99	do	3,532	211.4	164.8	17.8	14.3	12.9	12.1	----------
	2	99	Feb. 20, 1937	2,281	173.7	147.5	12.8	11.4	10.6	10.3	9.9
	6 [4]	[2] 43	Jan. 1, 1941	3,434	435.0	333.4	44.2	39.2	36.5	32.3	----------
	10	60	do	3,285	171.5	152.5	16.3	14.4	13.3	12.2	----------
	5	101	do	2,004	309.8	193.4	26.3	19.9	17.4	16.6	----------
	3	[2] 50	do	1,570	161.1	136.9	15.7	11.9	11.2	10.8	----------
	3	[2] 50	Aug. 6, 1939	1,444	183.7	139.9	54.4	12.5	11.4	11.1	10.6
	7	[2] 153	Jan. 1, 1941	955	159.0	128.5	30.3	14.8	13.1	12.7	----------
	8 [4]	[2] 201	do	814	291.1	224.8	49.4	33.4	29.0	26.5	----------
	4 [4]	[2] 42	do	711	606.0	485.5	80.0	59.5	53.9	50.6	----------
Oregon	5	99E	do	4,729	255.8	172.9	21.9	17.3	15.2	14.5	13.7
	5	99E	Jan. 1, 1940	4,054	285.2	167.9	26.4	15.7	14.6	13.9	12.8
	3	30	Nov. 27, 1937	1,252	248.5	175.2	29.2	17.6	16.6	15.6	13.7
	2	99	Jan. 1, 1940	915	237.8	157.0	24.4	18.4	16.3	14.9	13.9
Washington	4	99	Jan. 1, 1941	5,520	205.7	160.5	19.9	16.7	14.3	13.4	12.6
	4	99	Dec. 11, 1937	3,479	224.8	171.3	23.7	19.3	16.8	15.6	14.2
	1	99	Dec. 28, 1937	3,590	255.8	207.9	28.4	20.9	19.6	18.5	16.9
	3	101	Sept. 11, 1937	3,385	269.5	207.3	29.3	23.0	19.0	17.8	16.5
	10	10	Apr. 10, 1937	3,233	182.0	168.5	15.3	13.6	13.1	12.6	12.1

[1] Merritt Parkway.
[2] State routes. Others are U S numbered routes.
[3] County trunk highway.
[4] Traffic highly seasonal and not representative of most roads in the State. Not included in regional averages.

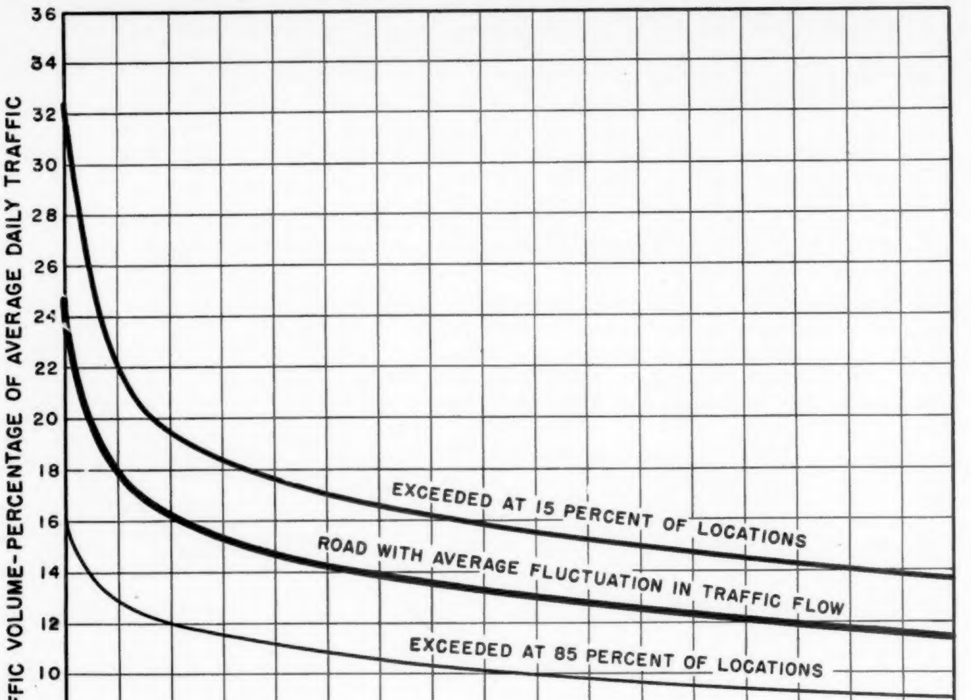


Figure 50.—Relation between peak hourly flows and annual average daily traffic on rural highways.

with the average fluctuation in traffic flow, the peak hourly volume is about 25 percent of the annual average daily volume. At 15 percent of the locations, however, the peak hourly volume during 1 year exceeded 32 percent of the average daily volume, whereas at another 15 percent of the locations the peak hourly volume was less than 16 percent of the average daily volume. At all locations, there are relatively few hours during the year when the traffic volume greatly exceeds the volume that occurs frequently or for a large number of hours each year. This would be more evident were the curves in figure 50 extended to include the 8,760 hours of a full year, in which case they would reach or approach the zero base line at a distance to the right equivalent to about 51 times the width of the chart.

Thirtieth Highest Hour a Practical Criterion of Needed Capacity

The relation between the peak hourly flows and the annual average daily traffic on rural highways, as shown by figure 50, serves as a guide when selecting a reasonable hourly volume for the design of a highway. Looking first at the curve showing the traffic fluctuation on the average rural road, it will be seen that the peak hour of the year will require a roadway having over twice the capacity that would be necessary if some degree of congestion could be tolerated during 130 hours of the year. To illustrate, consider a proposed highway for which an annual average daily volume of 4,200 vehicles is anticipated. To provide for the highest hourly volume during the year would require a capacity equal to 25 percent of 4,200, or 1,050 vehicles per hour. A facility capable of accommodating the traffic during all except the 130 highest hours of the year would require a capacity of only about 500 vehicles per hour.

During the highest 130 hours of the year, however, a very large number of drivers would be inconvenienced by congestion if the practical capacity were only 500 vehicles per hour. By planning for the volume during the 120th highest hour instead of the 130th, the change in the required design would be inconsequential because the difference in the two hourly volumes is negligible. Continuing with this procedure, **it will be found that at about the thirtieth highest hour, the slope of the curve changes rapidly, and it is at this point that the ratio of benefit to expenditure is near the maximum.**

Providing for an hourly traffic volume that is not exceeded at least 30 times a year will show an extremely small return in terms of driver benefit, whereas little will usually be saved in the construction cost and a great deal lost in driver benefit if provision is not made for the fiftieth highest hourly traffic volume of the year. It is for this reason that a design which will accommodate the fiftieth highest hourly traffic volume of the year can usually be justified, whereas a design to accommodate a traffic volume greater than that occurring during the thirtieth highest hour is generally not warranted.

Since this analysis is based on the average fluctuation in traffic volume for many highways, the results are not necessarily applicable to every location. However, by following a similar approach for the locations with high and low fluctuations in traffic flow, as represented by the upper and lower curves of figure 50, it can be shown that for these cases, and hence for most cases, the thirtieth highest hourly volume for the year is generally a reliable criterion of the needed capacity for which it is most practical to design. This criterion was recently adopted by the American Association of State Highway Officials as a design policy for the National System of Interstate Highways.

The strict adherence to the application of this criterion, however, will not always result in the best engineering practice. There are numerous specific locations where a design to accommodate some hourly volume other than the thirtieth highest of the year would be proper. For this reason, and because adequate traffic counts on which to base the design of a project are relatively inexpensive compared with the cost of construction, **the results of a continuous hourly traffic count for a full year should be available for locations at or near each place where a major construction or reconstruction project is contemplated.**

By considering the shape of the yearly traffic pattern—the curve found by arranging all the hourly volumes of one year in descending order of their magnitude—the most feasible hourly volume that should be used for design purposes can be determined.

In connection with the selection of the design volume, future increases in the traffic volume due to the normal increase in travel and induced traffic due to the improvement of a facility must be considered, together with the estimated life of the contemplated construction and the traffic capacities of the various possible designs. If, for example, the thirtieth highest hourly volume in 20 years would require a four-lane highway, whereas the fiftieth highest volume would only require a two-lane highway, it would be most practical to provide the two-lane highway using the fiftieth highest hourly volume for design purposes. If, however, a four-lane highway would be needed within 10 years, it may still be most feasible to construct only a two-lane road at the present time and make the necessary provision so that the two additional lanes can be constructed for a reasonable cost at some future date. In this case, the alinement

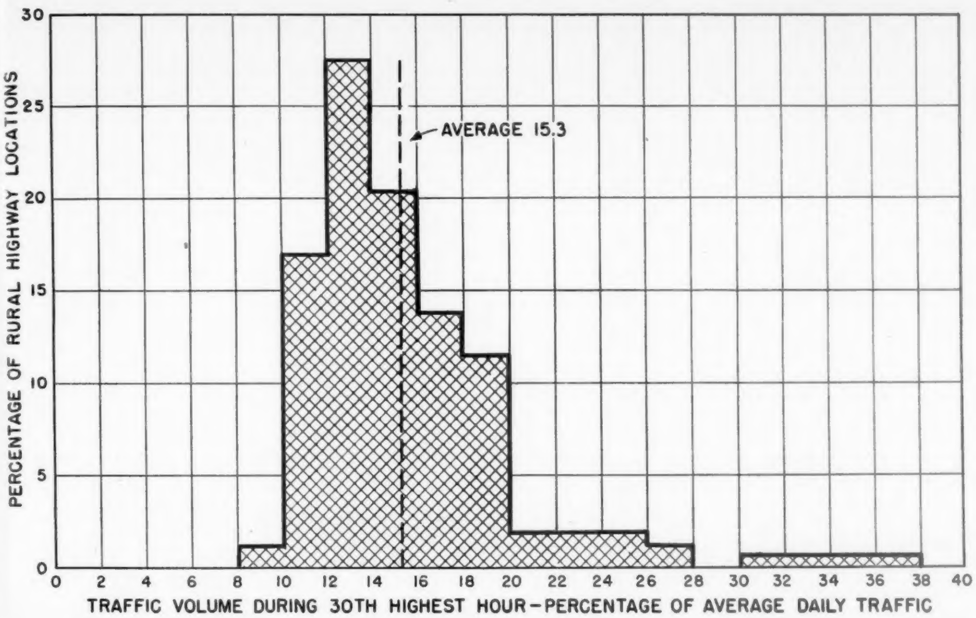


Figure 51.—Distribution of locations by percentage of average daily traffic during thirtieth highest hour of the year.

used might be such that the two-lane road would adequately serve traffic for a period of 10 years if in relatively flat terrain, whereas, if the road was located in rough terrain it might be more economical to build the two-lane road to the same alinement as a four-lane road (spending no funds to provide passing sight distances) even though the four-lane construction might then be required in less than 10 years.

Relation of Thirtieth Highest Hour to Annual Average Daily Traffic

The range in the relation between the thirtieth highest hourly volume and the annual average daily volume at rural locations is shown by figure 51. At nearly half of the locations, the thirtieth highest hour was between 12 and 16 percent of the annual 24-hour average. In view of the emphasis which has previously been placed on 10 percent as a peak-hour percentage, it is interesting to note that the thirtieth highest hour was under 10 percent at less than 2 percent of the locations and over 10 percent at 98 percent of the locations. Although it is true that on any single day the maximum hourly volume is likely to be close to 10 percent of the traffic on that day, **the use of a 10-percent value for design purposes is sound only where there is an exceptionally low seasonal and day-to-day variation in traffic flow.**

Only at relatively few locations is the thirtieth highest hourly volume an exceptionally high percentage of the annual average daily volume, as shown in figure 51 by the areas on the right which are above 20 percent. It might appear that these percentages are too high for use as design criteria and that they are probably too exceptional to be repeated year by year at the same locations. This is not the case, however, because the results of a study of the yearly patterns for different years at a number of locations (table 24) show that **the thirtieth highest hourly volume on a percentage basis changes very little from year to year.**

The consistency of the thirtieth highest hour as a part of the yearly traffic pattern is shown by figure 52. Data for two different years at 24 rural highway locations were summarized, with the thirtieth highest hour expressed as a percentage of the annual average 24-hour traffic volume. From this illustration **it is immediately evident that the many factors which operate to influence traffic at a particular location do not materially change the thirtieth highest hour percentage.** For example, locations 1 and 2 maintain their unusually high percentages despite the passage of 2 and 3 years, respectively. At location number 10 with a 39-percent increase in annual traffic over a 4-year period, the traffic volume in the thirtieth highest hour, expressed as a percentage of average daily traffic, changed from 16.0 to 16.2 percent. The comparison is just as favorable for lightly as for heavily traveled roads, as will be seen by noting the relative heights of the two bars for location 8 and those for location 20. In a few cases, average daily traffic volumes have dropped, as at locations 6, 12, and 16, but this has not resulted in any very marked change in the percentages representative of the thirtieth highest hours. For all 23 locations combined, there is an average difference of 1.1 percent between the

Table 24.—*Comparison of variations in traffic flow at the same location for different years*

State	Recorder station No.	Route No.	Year starting	24-hour volumes			Percentage of average 24-hour volume in certain hourly volumes during year				
				Average for year	Percentage of average in— Maximum 24 hours	Percentage of average in— Tenth highest 24 hours	Maximum hour	Tenth highest hour	Twentieth highest hour	Thirtieth highest hour	Fiftieth highest hour
				Vehicles	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
Rhode Island	2	1	June 4, 1938	1,931	466.8	238.0	68.3	43.8	37.9	34.2	31.4
			Jan. 1, 1940	2,155	452.9	263.9	57.2	41.6	37.1	34.0	30.0
New Hampshire	1	3	Sept. 18, 1937	1,360	647.6	279.9	60.4	48.4	38.7	34.1	28.9
			Jan. 1, 1940	1,410	517.2	277.8	55.2	43.0	37.5	32.8	27.5
Michigan	678	23	Jan. 1, 1939	1,200	300.9	267.5	44.8	31.2	26.2	24.8	22.5
			Jan. 1, 1941	1,538	336.0	246.8	36.4	28.3	26.0	24.3	23.0
Utah	301	40	Nov. 13, 1937	1,766	301.1	260.0	41.3	29.0	26.2	25.4	22.8
			Jan. 1, 1941	2,111	355.0	247.7	29.2	26.4	23.1	21.9	19.6
Vermont	A-12-2	2	Nov. 28, 1936	1,615	276.5	219.6	31.3	22.2	20.9	20.3	18.0
			Jan. 1, 1940	1,688	236.3	196.1	38.1	19.5	17.5	16.9	16.2
Maine	2	1	Feb. 5, 1938	1,287	274.1	263.4	22.4	19.8	18.8	17.9	17.1
			Jan. 1, 1940	1,248	274.3	231.6	24.0	20.6	19.5	18.7	17.6
Wisconsin	3-EB	41 [1]	Jan. 8, 1938	2,857	281.6	220.3	22.1	19.7	17.8	16.4	15.3
			Jan. 1, 1941	3,921	439.8	217.0	30.4	23.4	20.5	18.9	16.0
New Mexico	9	54	Jan. 8, 1938	751	241.5	165.6	30.5	20.4	17.0	16.4	14.8
			Jan. 1, 1941	946	239.6	187.9	28.9	19.2	18.0	16.6	15.8
Wyoming	204	30	Jan. 1, 1939	1,257	234.4	197.9	26.6	18.8	17.1	16.5	15.0
			Jan. 1, 1941	1,647	233.3	198.7	31.8	17.7	16.6	15.8	14.3
Pennsylvania	1	20	Nov. 20, 1937	4,395	311.1	209.6	28.1	19.3	17.2	16.0	14.9
			Jan. 1, 1941	6,112	291.3	212.7	23.9	20.0	17.2	16.2	15.3
Washington	4	99	Dec. 11, 1937	3,479	224.8	171.3	23.7	19.3	16.8	15.6	14.2
			Jan. 1, 1941	5,520	205.7	160.5	19.9	16.7	14.3	13.4	12.6
Utah	302	50	July 10, 1937	3,443	230.9	172.4	20.9	16.9	15.1	14.4	13.7
			Jan. 1, 1941	4,422	231.5	164.4	17.7	15.4	14.5	14.2	13.4
Oregon	5	99E	Jan. 1, 1940	4,054	285.2	167.9	26.4	15.7	14.6	13.9	12.8
			Jan. 1, 1941	4,729	255.8	172.9	21.9	17.3	15.2	14.5	13.7
Colorado	14	160	Jan. 1, 1940	1,286	327.9	151.2	46.0	18.4	14.7	14.5	12.4
			Jan. 1, 1941	1,224	319.9	160.0	46.1	17.4	15.9	13.6	12.4
Iowa	601	65	Dec. 19, 1936	3,290	232.4	154.2	18.1	14.9	14.3	13.4	12.3
			Jan. 1, 1938	3,539	223.3	157.4	26.7	16.6	14.9	14.2	13.2
New Mexico	1	85	June 12, 1937	1,216	238.3	184.8	23.0	17.2	16.0	14.9	13.7
			Jan. 1, 1941	1,059	192.8	168.1	28.0	14.8	13.9	13.2	12.7
Texas	4	77	Jan. 1, 1938	4,049	151.9	143.5	16.6	14.2	13.3	12.8	12.0
			Jan. 1, 1941	5,180	181.2	145.9	19.4	14.6	13.3	12.9	12.0
California	1	99	July 10, 1937	5,815	197.2	159.9	18.9	15.3	13.5	12.4	10.8
			Jan. 1, 1941	7,692	207.0	149.3	17.1	13.7	12.8	12.2	11.0
Wyoming	205	20	May 19, 1939	1,309	230.2	149.7	54.7	15.0	13.7	13.1	12.3
			Jan. 1, 1941	1,375	216.7	152.7	20.1	13.3	11.8	11.0	9.5
Arizona	1	80	July 7, 1939	7,174	172.1	131.2	22.9	13.0	11.9	11.5	10.9
			Jan. 1, 1941	8,757	157.4	130.1	16.3	12.2	11.7	11.2	10.5
California	2	99	Feb. 20, 1937	2,281	173.7	147.5	12.8	11.4	10.6	10.3	9.9
			Jan. 1, 1941	3,532	211.4	164.8	17.8	14.3	12.9	12.1	----
California	3	50 [1]	Aug. 6, 1939	1,444	183.7	139.9	54.4	12.5	11.4	11.1	10.6
			Jan. 1, 1941	1,570	161.1	136.9	15.7	11.9	11.2	10.8	----
Florida	4	90	May 15, 1937	3,365	144.2	119.7	14.9	10.8	10.3	9.8	9.2
			Jan. 1, 1941	3,646	147.3	132.1	13.3	11.9	11.3	10.9	10.3

[1] State routes. Others are U S numbered routes.

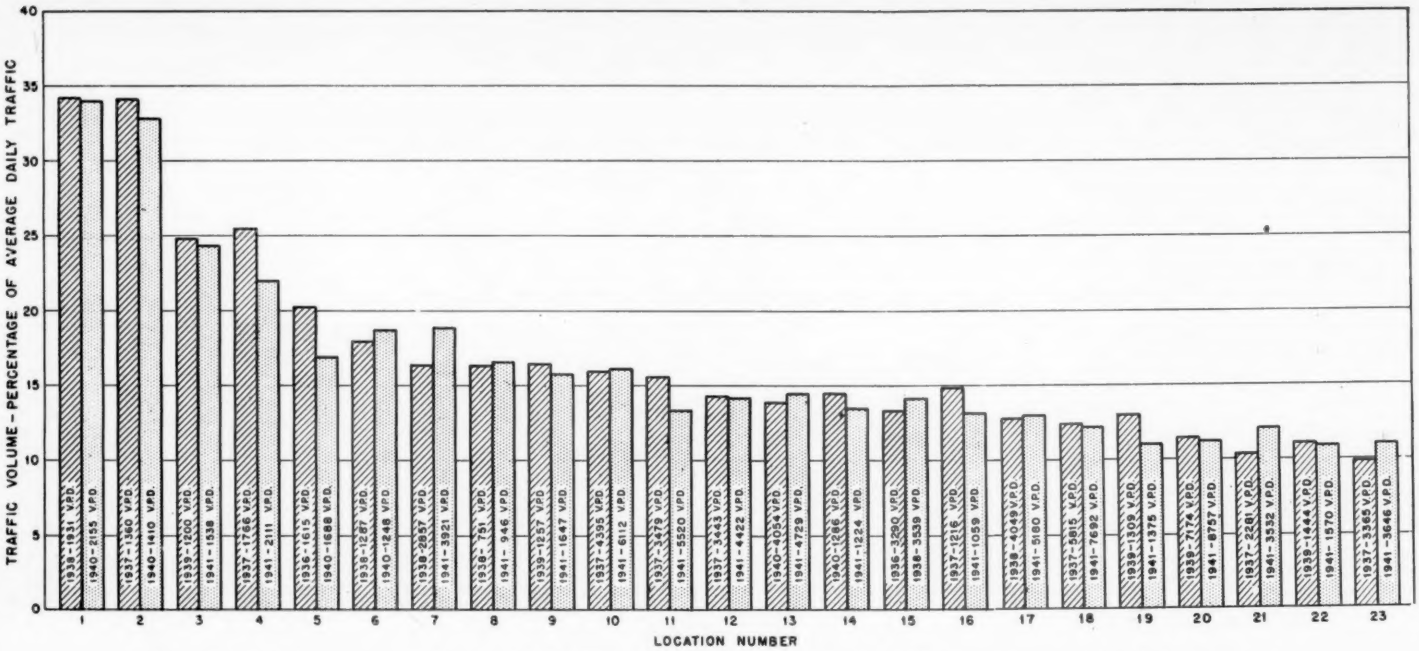


Figure 52.—Comparison of thirtieth highest hourly volumes for two different years at idential locations.

thirtieth highest hour percentages for different years at the same location.

The fact that there is this invariability in the thirtieth highest hour at a given location adds greatly to its worth as a design criterion, since with a given annual average daily volume for some future year obtained through forecasts, it is at once possible to compute with considerable confidence the traffic load on the facility during the thirtieth highest hour of that future year. For example, if conditions indicate that a 20-percent rise in the annual average may be expected in 10 years, a similar 20-percent increase should be expected in the thirtieth highest hour; that is, if the facility is able to handle that much traffic. If it is not, severe congestion is apt to result and traffic will either use other routes or the drivers will be subject to long delays.

If the facility is already partially congested during 30 hours of a year, a 20-percent increase in total traffic will worsen the congestion far more than this proportionate amount. A 20-percent gain in annual average daily traffic will increase the number of hours of congestion from 30 to approximately 100 hours, a gain of 333 percent. Furthermore, the average congestion during the 100 hours will be much more than during the 30 hours of the previous year. Thus at an average location, if the average daily volume is 19,600 vehicles the thirtieth highest hour is 3,000 vehicles per hour. Should traffic increase 20 percent during the next 10 years, or to 23,520 vehicles per day, the thirtieth highest hour during that year will be higher in direct proportion—specifically, 3,600 vehicles per hour—if, as previously stated, the facility is capable of handling this larger volume. A volume of 3,000 vehicles per hour will then be 12.5 percent of the average daily traffic which, as may be seen from figure 50, is exceeded during about 100 hours of the year.

Table 25.—*Variations in traffic flow on major urban facilities during 1 year*

City and location	Type [1] of facility	24-hour volume			Percentage of average 24-hour volume in certain hourly volumes during year				
		Average for year	Percentage of average in—						
			Maximum 24 hours	Tenth highest 24 hours	Maximum hour	Tenth highest hour	Twentieth highest hour	Thirtieth highest hour	Fiftieth highest hour
		Vehicles	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
Birmingham, Ala.: Roebuck Blvd.	A-O	6,742	155.9	143.7	17.2	15.5	14.7	13.8	9.9
Chicago, Ill.:									
Leif Erikson Dr	E-I	41,590	137.9	129.0	12.6	11.2	10.7	10.4	10.1
Michigan Ave	A-D	69,736	131.9	118.2	10.0	8.9	8.3	8.2	7.9
Monroe St	A-D	32,102	140.0	124.1	11.5	9.6	9.2	8.9	8.6
Ashland Blvd	A-I	16,919	129.0	114.6	10.4	9.9	9.6	9.5	9.3
Jackson Blvd	A-I	20,939	133.3	122.1	11.3	10.2	9.8	9.6	9.4
Sacramento Blvd	A-I	13,243	142.5	122.5	13.5	12.3	11.9	11.8	11.5
Warren and Washington Blvds.	A-I	39,374	138.4	123.8	12.9	12.1	11.3	10.3	9.9
Lake Shore Dr	E-I	85,698	140.7	124.6	13.5	11.5	10.9	10.7	10.3
Detroit, Mich.:									
Joy Rd	I	10,784	139.6	129.9	15.2	11.8	11.3	11.1	10.6
Six Mile Rd		22,768	124.5	117.3	11.1	9.8	9.8	9.7	9.4
14th St. at Edison		12,894	140.6	122.6	15.3	12.4	12.0	11.8	11.7
Albuquerque, N. Mex.: North 4th St.	A-O	3,375	173.7	146.2	15.2	13.7	13.2	12.8	12.4
Santa Fe, N. Mex.: Don Gaspar St.	A-D	4,679	166.1	133.7	18.6	12.6	12.1	11.8	11.3
New York, N. Y.: George Washington Bridge.	E-O	22,000	245.6	212.4	22.5	18.8	17.8	16.9	11.6
Philadelphia, Pa.:									
Chestnut St. Bridge [2]	E-D	30,200	129.7	120.1	8.7	8.1	8.0	7.3	6.9
Parkway and 22d St	A-D	51,200	113.6	112.4	11.5	11.4	11.1	11.0	10.8
Spring Garden Bridge	A-I	19,500	122.5	118.2	16.6	12.7	12.4	11.7	11.0
Girard Ave. Bridge	A-I	43,800	120.2	117.1	11.1	10.6	10.2	10.0	9.8
Wissahicken and Ridge Sts	A-O	40,500	119.1	114.0	11.9	10.4	10.1	9.6	9.4
City Line Bridge	E-O	24,360	148.4	138.7	13.1	11.7	11.4	10.7	10.2
Allegheny and Hunting Park	A-O	29,500	129.9	118.8	10.6	10.1	10.0	9.7	9.4
Broad, Glenwood, Cambria	A-I	51,000	122.3	118.8	9.3	9.2	9.0	8.8	8.6
5th and Roosevelt Blvd	A-O	23,400	151.1	145.4	16.6	14.3	13.6	13.6	13.3
Ogontz and Olney Ave	A-O	19,670	124.2	120.9	12.9	10.7	10.5	10.3	10.1
Washington, D. C.:									
Fourteenth St. Bridge	A-I	41,300	138.5	115.7	9.6	9.2	9.0	8.8	8.2
Memorial Bridge	E-I	36,700	151.8	116.9	14.7	13.0	12.4	12.1	11.9
Key Bridge	A-I	32,600	143.0	117.6	11.3	9.7	9.7	9.4	8.8
Anacostia Bridge	A-O	32,278	114.0	104.4	9.0	8.3	8.1	7.9	7.3
Benning Rd. NE	A-O	27,725	141.6	115.6	12.8	9.1	8.7	8.5	7.5
Bladensburg Rd. NE	A-O	27,123	138.6	107.9	10.4	9.7	9.1	9.0	8.5
Connecticut Ave	A-I	26,842	116.7	110.0	9.1	8.8	8.5	8.0	7.2
Pennsylvania Ave	A-I	24,388	123.1	112.4	8.8	8.3	7.9	7.7	6.6
Georgia Ave. NW	A-O	21,628	125.7	117.1	10.4	9.1	8.7	8.7	8.1
Wisconsin Ave	A-O	20,786	129.4	111.8	10.4	10.2	9.7	9.5	9.2
Rhode Island Ave. NW	A-I	19,695	117.6	103.8	9.3	8.8	8.5	8.3	7.9
13th St. NW	C-D	16,857	121.7	106.9	10.6	9.6	9.4	9.1	8.1
K St. NW	A-D	15,618	115.2	109.1	10.7	10.4	10.0	9.9	8.9
Total		28,329	136.8	122.6	12.4	10.9	10.5	10.2	9.5

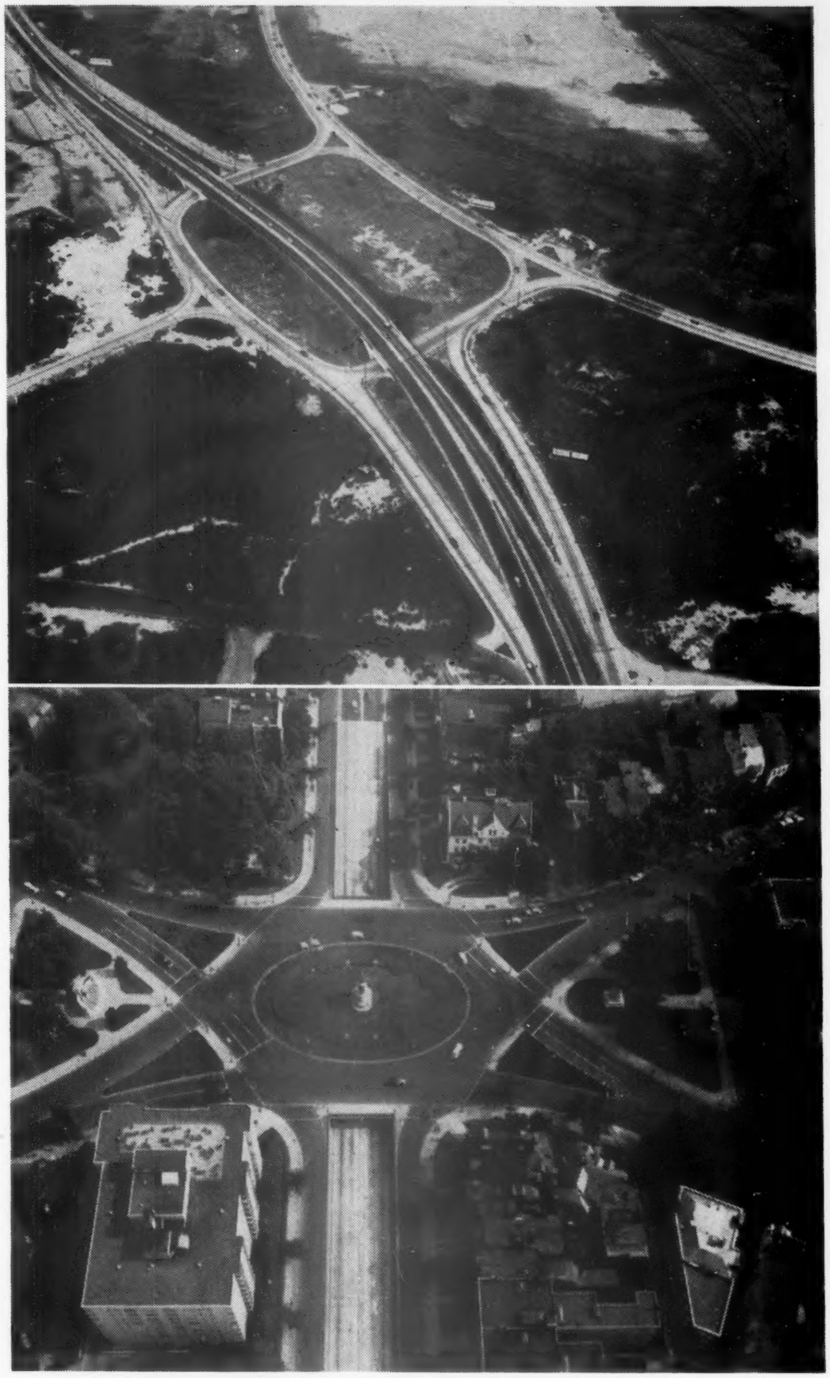
[1] Type of facility code:
E=Expressway. O=Outlying.
A=Arterial. I=Intermediate.
C=City Street. D=Downtown.

[2] One-way.

Peak-Hour Traffic in Urban Areas

Though most of the foregoing has dealt with the conditions on rural highways, more and more facts similar to those presented in table 25 are becoming available for urban facilities. More comprehensive counts are needed, however, before anything like a complete presentation of the relationships between peak and average-hour volumes can be made for city conditions. The data in table 25 do show, nevertheless, that peak hours on the average city facility are a somewhat lower percentage of the annual average 24-hour traffic volume than is the case for rural highways. It should be noted that there are numerous exceptions in the table to this generalization, principally on those streets or highways that serve suburban or outlying areas. While there are insufficient data presently available to permit full explanation of this difference between the thirtieth peak hour relationship in downtown and rural areas, a fact which may well be considered is that **many downtown facilities are severely congested during hundreds of hours of the year.** It is on this type of facility that traffic counters are generally installed. **Peak-hour volumes that otherwise might be recorded several times during the year are impossible of attainment because the facility is incapable of accommodating traffic in excess of the load imposed upon it almost every day during the hours of heaviest demand.** The exceptionally heavy peak load, therefore, must be distributed over alternate routes that are usually more circuitous and less desirable in some other respects than the arterial, were it not congested.

If proposed urban improvements are to provide satisfactory service, the percentage of the average daily traffic used for the design peak hour should be ascertained by a study of facilities that are capable of absorbing some overload during all hours of the year. Arterial streets of this kind are very rare, indeed, under present-day conditions. Until such time as more comprehensive data become available, a percentage for traffic moving in the thirtieth highest hour approximating or only slightly lower than the percentage observed in rural areas is recommended for the design of urban facilities.



Traffic circles take many forms: Shown here are tangent and circular sections for weaving. When traffic circles are no longer able to accommodate their traffic loads the necessary remedial measures, such as an added overpass (above) or tunnel (below), may cost several million dollars.

Effect of Directional Distribution of Traffic

From the discussions in parts III and IV it will be recalled that the capacities of two- and three-lane roads must be in terms of total traffic. The one-direction capacity of these facilities is dependent upon the number of vehicles approaching from the opposite direction, and the hour of the day when the two-directional movement is at a maximum constitutes the period of greatest congestion. Hence the foregoing discussion relating to the percentage of the average daily traffic in the thirtieth highest hour has taken account only of the total traffic moving in both directions. On all multilane facilities, however, and on two- and three-lane roads where improvement to the multilane type is contemplated, consideration of the directional traffic load may be of superior importance. For example, each lane of a four-lane divided facility may be capable of handling 1,500 vehicles per hour, a nominal total of 6,000 vehicles per hour. However, peak-hour traffic flows are not evenly balanced by direction except in very rare instances. A frequent condition on rural highways, for example, is a peak flow of two-thirds in one direction and one-third in the other direction. Consequently, the facility in question would carry only a total of 4,500 vehicles when the capacity of the heavy direction is completely absorbed. Obviously, more than four lanes would be needed to accommodate a total flow of 6,000 vehicles per hour under these circumstances.

As an aid when considering the variations in traffic flow by direction on multilane roadways, tables 26 and 27 show the percentage that the thirtieth highest hour volume is of the annual average 24-hour traffic volume in one and in both directions for a number of rural and urban facilities. Of more significance than the actual percentages for the few rural and urban facilities included in these tables is the relation between the one-direction

Table 26.—*Comparison of variations in total traffic flow with variations in one direction of travel*

State	Recorder station No.	Route No.	Year starting	24-hour volume			Percentage of average 24-hour volume in certain hourly volumes during year				
				Average for year	Percentage of average in— Maximum 24 hours	Percentage of average in— Tenth highest 24 hours	Maximum hour	Tenth highest hour	Twentieth highest hour	Thirtieth highest hour	Fiftieth highest hour
				Vehicles	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
BOTH DIRECTIONS COMBINED:											
New Jersey (Essex County)	NB and SB	25	Jan. 1, 1941	62.250	143.6	131.6	11.2	9.6	9.3	9.3	8.8
New Jersey (Edison Bridge)	NB and SB	35	do	25,381	284.3	231.6	24.0	19.1	17.3	16.2	15.4
New Jersey (Woodbridge)	NB and SB	35	do	22,052	229.2	211.5	26.8	18.0	17.0	15.9	13.6
Connecticut	6 and 7	1	Mar. 31, 1939	13,624	292.5	227.9	25.7	21.2	19.2	18.6	17.7
Delaware	C	13	June 8, 1941	6,741	254.2	206.5	23.8	20.5	20.0	19.1	16.9
Wisconsin	2 and 3	[1] 41	Jan. 8, 1938	5,674	305.9	220.2	29.1	22.2	18.9	18.1	16.3
ONE DIRECTION:											
New Jersey (Essex County)	SB	25	Jan. 1, 1941	30,150	159.9	137.8	13.3	11.4	11.1	10.9	10.3
New Jersey (Edison Bridge)	SB	35	do	11,656	436.3	315.3	36.0	28.0	24.7	24.7	23.6
New Jersey (Woodbridge)	SB	35	do	10,766	489.0	255.3	41.5	30.1	26.2	21.7	18.9
Connecticut	6–WB	[2] 1	Mar. 31, 1939	6,813	287.7	244.3	29.7	25.6	24.0	23.4	22.0
Delaware	C–NB	13	June 8, 1941	3,371	344.9	235.9	40.1	34.6	31.4	29.2	25.6
Wisconsin	2–WB	[1] 41	Jan. 8, 1938	2,817	405.6	241.8	43.7	30.7	25.6	22.9	20.2

[1] U S numbered routes. Others are State routes.
[2] Merritt Parkway.

and two-direction percentages. It will be seen that there are but few cases where the percentage of the average daily traffic moving in one direction during the thirtieth highest hour of the year is not substantially higher than the percentage for both directions. One reason for this, of course, is that the peak volumes in one direction do not always occur at the same time as the peak total volumes. The latter are more apt to occur on multilane facilities when there is a fairly high traffic movement in both directions.

As a practical matter, it is much more likely that the engineer would have information only on total traffic movement, especially for two- or three-lane roads. To illustrate the manner in which the information from table 27 might be employed, let it be assumed that the present annual average daily traffic on a two-lane rural road is 4,800 vehicles per day

Table 27.—*Thirtieth highest hour traffic volume during the year as a percentage of the annual average 24-hour traffic volume in one and in both directions on multilane rural and urban highways and streets*

Location	Percentage of annual average daily traffic in thirtieth highest hour of year	
	Both directions	One direction
RURAL FACILITIES:		
Connecticut (Merritt Parkway)	18.6	23.4
New Jersey:		
U S 1 near Newark	9.3	10.9
Edison Bridge, State Route 35	16.2	24.7
Woodbridge St., Route 35	15.9	21.7
Delaware	19.1	29.2
Wisconsin	18.1	22.9
Rural averages	16.2	22.1
URBAN FACILITIES:		
Chicago (Parkways)[1]	9.9	14.1
New York (George Washington Bridge)	16.9	24.9
Philadelphia [2]	10.6	13.1
Washington, D. C.[3]	9.0	11.9
Urban averages	11.6	16.0

[1] 8 locations.
[2] 9 locations.
[3] 13 locations.

and, in that locality, the thirtieth highest hourly volume of the year on roads that are not badly congested is 16.2 percent of the average daily traffic.

Assume also that it is desired to calculate the capacity of the road that will be needed 10 years hence when the estimated traffic for the design year, including the immediate increase on the improved facility, will be 150 percent of 4,800 or 7,200 vehicles per day. The thirtieth highest hourly volume will be 16.2 percent of 7,200 vehicles, or 1,166 vehicles per hour. This exceeds the practical capacity of a two-lane road even with the best alinement. Furthermore, sight-distance restrictions and other limitations imposed by the terrain might lower the practical capacity of a two-lane road at this location to some value considerably below 900 passenger cars per hour, the practical capacity of a good two-lane road. The required capacity for one direction of a four-lane road would be 22.1 percent (table 27) of 3,600 vehicles, or 796 vehicles per hour. This same value may be approximated by assuming that two-thirds of the thirtieth highest hourly volume of 1,166 vehicles will be traveling in the one direction.

Although conditions differ rather widely among facilities, **it is usually sound, in the absence of detailed information as to traffic volume balances, to assume that two-thirds of the traffic will be in one direction during the design peak hour in rural areas and in outlying urban areas. As the central business districts of large cities are approached, the distribution of traffic by directions usually becomes more evenly balanced, sometimes closely approaching a 50–50 ratio in the central business area.**

A tabulation of traffic by direction of movement for the streets reported in the intersection study shows that in downtown areas an average of 55 percent of the traffic on the congested streets was moving in the heavier direction, and that for 70 percent of these streets between 50 and 55 percent was in this direction of heavier movement. The average directional distribution of traffic on streets in intermediate areas was 61.6 percent in the heavier direction during peak hours with 38.4 percent in the direction of lesser movement. In outlying areas the distribution was 65.6 percent in the heavier direction and 34.4 percent in the other direction.

Traffic During Various Periods of the Day

The information shown in table 28 will be of some assistance to an engineer considering a proposed improvement when the only available traffic data consist of a few short counts. In some cases, by relating the available counts to the information shown in tables 22–27, it will be possible to obtain a fairly accurate estimate of the practical capacity that should be provided or the excess capacity available on an existing facility. **Such a procedure is not recommended, however, and should not be condoned for any project involving extensive improvements when it is still possible to obtain complete traffic information before preparing final plans for the improvements.**

On main rural highways, an average of 74 percent of the day's traffic occurs between 7 a. m. and 7 p. m., with at least 70 percent of the locations represented coming within 5 percent of this value. Local routes in rural areas have a slightly higher percentage of the total day's traffic during this period than the main routes, whereas most urban facilities carry less than 70 percent of their traffic between 7 a. m. and 7 p. m.

Though the 2 p. m. to 10 p. m. and 8 a. m. to 4 p. m. periods each account for approximately half of the total day's traffic on rural routes, the 2 p. m. to 10 p. m. period is definitely the heavier part of the day in urban areas.

The traffic volume during the 8 hours (not necessarily consecutive) that have the highest total flow is of special interest in connection with urban problems since this base is commonly used in making counts for traffic signals and similar improvements. As an illustration of the need for this type of information, the recently revised Manual on Uniform Traffic Control Devices specifies that the total vehicular volume entering an intersection from

Table 28.—*Percentage of traffic during various periods of the day*

Type of highway and location	Percentage of 24-hour volume in the period—											
	6 a. m. to 2 p. m.		2 p. m. to 10 p. m.		10 p. m. to 6 a. m.		8 a. m. to 4 p. m.		7 a. m. to 7 p. m.		Highest 8 hours (not consecutive)	
	Average	Range	Average	Range	Average	Range	Average	Range	Average	Range	Average	Range
RURAL CONDITIONS: [1]												
Main rural highways	41	38-44	49	46-52	10	8-13	49	44-55	74	69-79	57	53-60
Local rural routes	45	41-50	48	44-50	7	5- 9	52	50-58	78	75-82	59	53-65
URBAN CONDITIONS: [2]												
Washington, D. C. (weekdays):												
13 locations	40	38-42	47	46-50	13	10-15	41	36-47	70	65-74	52	49-60
In-bound	45	40-56	43	39-46	12	9-15	44	37-51	70	64-76	53	50-62
Out-bound	34	38-37	52	48-60	14	11-17	38	32-43	69	63-73	53	48-60
Chicago Parkways (weekdays):												
8 locations	38	34-46	48	45-49	16	12-21	44	38-49	68	61-72	55	50-61
In-bound	49	35-54	39	30-44	12	9-16	55	49-60	74	67-79	58	52-63
Out-bound	25	15-35	56	48-62	19	13-26	33	21-45	64	55-72	58	50-65
Detroit, Mich.: 3 locations	39	38-41	48	47-50	12	11-14	40	38-42	70	67-72	54	53-55
I elaware River Bridge:												
Maximum weekday	35	----	44	----	21	----	40	----	62	----	49	----
To Philadelphia	38	----	37	----	25	----	41	----	59	----	49	----
To Camden	31	----	51	----	18	----	39	----	65	----	55	----
Maximum Sunday	26	----	46	----	28	----	32	----	50	----	52	----
To Philadelphia	9	----	55	----	36	----	14	----	32	----	72	----
To Camden	37	----	35	----	18	----	54	----	71	----	54	----
New York City bridges and tunnels:												
Weekdays	--	----	--	----	--	----	--	----	71	----	--	----
Saturdays	--	----	--	----	--	----	--	----	67	----	--	----
Sundays	--	----	--	----	--	----	--	----	64	----	--	----

[1] Range includes the averages for at least 70 percent of the locations
[2] Range includes the averages for all locations.

all approaches must average at least 750 vehicles per hour for any 8 hours of an average day before the vehicular volume alone will

Table 29.—*Annual average 24-hour volume and corresponding peak hourly volumes warranting a traffic signal, as based on Nation-wide traffic pattern and traffic-signal warrant from the Manual on Uniform Traffic Control Devices*

	Total traffic volume from all approaches at—	
	Urban intersection	Rural intersection
Annual average 24-hour volume	11,300	7,000
Peak hourly volumes:		
Highest hour	1,402	1,744
Tenth highest hour	1,232	1,246
Thirtieth highest hour	1,152	1,072
Fiftieth highest hour	1,074	988

warrant a fixed-time signal in an urban area. For rural areas, the corresponding criterion is 500 vehicles per hour.

Table 28 shows that 57 percent of the day's traffic on main rural highways and about 53 percent of the day's traffic at urban locations occur during the highest 8 hours of the average day. A fixed-time traffic signal, therefore, would not be warranted by the traffic volume alone unless the average annual volume from all approaches to the intersection exceeded 11,300 vehicles per day at an urban location, or 7,000 vehicles per day at a rural location.

Based on the average yearly traffic patterns for rural roads and urban facilities, as shown by tables 22 and 24, these annual volumes correspond to the peak hourly volumes shown in table 29.

Based on the results of the intersection studies included in part v of this report, and assuming the usual condition that two-thirds of the traffic on each highway will be moving in one direction during the peak hours, the total traffic through an intersection with one approach on each road loaded to its possible capacity will be as shown in table 30. With proper signal timing, the total intersection volume will be independent of the distribution of traffic between the two roads.

It is therefore apparent that the traffic volume at the intersection of two two-lane rural roads or at the intersection of two 40-foot city streets with parking must approach and in some cases exceed the possible capacities of the average intersection of the same type with traffic signals before the traffic volume warrants alone would justify the installation of traffic signals. It is also apparent that fixed-time signals on two-lane rural highways cannot be justified by the traffic volume warrant until the volume of traffic on such roads at times exceeds their practical capacities, especially when one of the intersecting roads carries about 75 percent of the total traffic at the intersection. These are important considerations in view of the part that traffic signals play in increasing the capacity of intersections at grade by helping to eliminate confusion and serious accidents.

Table 30.—*Total hourly volume through intersections of various types when one approach on each of the intersecting roads is operating at its possible capacity*

Type of intersection	Total hourly intersection volume with one approach on each road operating at its possible capacity, based on—	
	Possible capacity of average intersection of the same type	Highest capacities observed
Two two-lane rural roads	1,500	1,800
Two 40-foot city streets, with parking	1,125	1,800
Two 40-foot city streets, without parking	2,140	3,750

The Financing of Highways by Counties and Local Rural Governments, 1931–1941

The results of an extensive long-term study by the Bureau of Public Roads are reported in a new publication, *The Financing of Highways by Counties and Local Rural Governments, 1931–1941.* The report presents a discussion, and detailed statistical data, concerning the financing of highways by county and local rural governments during the 11-year period. Included is information for each year, by States, on county and local receipts, expenditures, and debt for rural highways, which has long been in demand but never available heretofore.

The publication was made possible by the collection of the basic data through the intensive effort of the State highway departments, the county and local governments, and the field offices of the Bureau of Public Roads; and the analysis and presentation were the work of the Financial and Administrative Research Branch, Bureau of Public Roads.

The publication is for sale, at 45 cents a copy, by the Superintendent of Documents, U. S. Government Printing Office, Washington 25, D. C., to whom all orders should be sent. Prepayment is required. The Bureau of Public Roads cannot undertake free distribution of the publication.

Bibliography on Roadside Control

The Bureau of Public Roads has recently issued a processed publication, *Bibliography on Roadside Control,* in which are included a general reference section on the title subject and special sections on outdoor advertising, set-back regulations, and roadside zoning. The listings are arranged by States, and chronologically beginning with 1930. A brief digest of the cited reference accompanies each listing.

The bibliography is intended to serve as an aid to highway planners and administrators in their activities dealing with the problem of roadside control. It was compiled by the Land Studies Section, Financial and Administrative Research Branch of the Bureau.

The publication is available to highway engineers and administrators for official use, and may be obtained by those so qualified by writing to the Bureau of Public Roads, Washington 25, D. C.

U. S. GOVERNMENT PRINTING OFFICE: 1949

A complete list of the publications of the Bureau of Public Roads, classified according to subject and including the more important articles in PUBLIC ROADS, may be obtained upon request addressed to Bureau of Public Roads, Washington 25, D. C.

PUBLICATIONS of the Bureau of Public Roads

The following publications are sold by the Superintendent of Documents, Government Printing Office, Washington 25, D. C. Please do not send orders to the Bureau of Public Roads.

ANNUAL REPORTS

(See also adjacent column)

Reports of the Chief of the Bureau of Public Roads:

1931, 10 cents.	1934, 10 cents.	1937, 10 cents.
1932, 5 cents.	1935, 5 cents.	1938, 10 cents.
1933, 5 cents.	1936, 10 cents.	1939, 10 cents.

Work of the Public Roads Administration:

1940, 10 cents.	1942, 10 cents.	1947, 20 cents.
1941, 15 cents.	1946, 20 cents.	1948, 20 cents.

HOUSE DOCUMENT NO. 462

Part 1 . . . Nonuniformity of State Motor-Vehicle Traffic Laws. 15 cents.

Part 2 . . . Skilled Investigation at the Scene of the Accident Needed to Develop Causes. 10 cents.

Part 3 . . . Inadequacy of State Motor-Vehicle Accident Reporting. 10 cents.

Part 4 . . . Official Inspection of Vehicles. 10 cents.

Part 5 . . . Case Histories of Fatal Highway Accidents. 10 cents.

Part 6 . . . The Accident-Prone Driver. 10 cents.

UNIFORM VEHICLE CODE

Act I.—Uniform Motor-Vehicle Administration, Registration, Certificate of Title, and Antitheft Act. 10 cents.

Act II.—Uniform Motor-Vehicle Operators' and Chauffeurs' License Act. 10 cents.

Act III.—Uniform Motor-Vehicle Civil Liability Act. 10 cents.

Act IV.—Uniform Motor-Vehicle Safety Responsibility Act. 10 cents.

Act V.—Uniform Act Regulating Traffic on Highways. 20 cents.

Model Traffic Ordinance. 15 cents.

MISCELLANEOUS PUBLICATIONS

No. 265T . . . . Electrical Equipment on Movable Bridges. 40 cents.

No. 191MP . . . Roadside Improvement. 10 cents.

No. 272MP . . . Construction of Private Driveways. 10 cents.

No. 1486D . . . Highway Bridge Location. 15 cents.

Highway Accidents. 10 cents.

The Taxation of Motor Vehicles in 1932. 35 cents.

Guides to Traffic Safety. 10 cents.

An Economic and Statistical Analysis of Highway-Construction Expenditures. 15 cents.

Highway Bond Calculations. 10 cents.

Transition Curves for Highways. $1.25.

Highways of History. 25 cents.

Public Land Acquisition for Highway Purposes. 10 cents.

The Financing of Highways by Counties and Local Rural Governments, 1931–41. 45 cents.

House Document No. 249. Highway Needs of the National Defense. 50 cents.

Highway Practice in the United States of America. 50 cents.

Public Control of Highway Access and Roadside Development (1947 revision). 35 cents.

Tire Wear and Tire Failures on Various Road Surfaces. 10 cents.

Legal Aspects of Controlling Highway Access. 15 cents.

House Document No. 379. Interregional Highways. 75 cents.

Highway Statistics, Summary to 1945. 40 cents.

Highway Statistics, 1945. 35 cents.

Highway Statistics, 1946. 50 cents.

Highway Statistics, 1947. 45 cents.

Principles of Highway Construction as Applied to Airports, Flight Strips, and Other Landing Areas for Aircraft. $1.50.

Federal Legislation and Regulations Relating to Highway Construction. 40 cents.

Manual on Uniform Traffic Control Devices for Streets and Highways. 50 cents.

Specifications for Construction of Roads and Bridges in National Forests and National Parks (FP-41). $1.25.

Single copies of the following publications may be obtained free upon request addressed to the Bureau of Public Roads. They are not sold by the Superintendent of Documents.

ANNUAL REPORTS

(See also adjacent column)

Public Roads Administration Annual Reports:

1943. 1944. 1945.

MISCELLANEOUS PUBLICATIONS

Road Work on Farm Outlets Needs Skill and Right Equipment.

Indexes to PUBLIC ROADS, volumes 17–23, inclusive.

Bibliography on Highway Lighting.

Bibliography on Highway Safety.

Bibliography on Automobile Parking in the United States.

Express Highways in the United States: a Bibliography.

Bibliography on Land Acquisition for Public Roads.

Bibliography on Roadside Control

REPORTS IN COOPERATION WITH UNIVERSITY OF ILLINOIS

No. 313 . . . Tests of Plaster-Model Slabs Subjected to Concentrated Loads.

No. 332 . . . Analyses of Skew Slabs.

No. 345 . . . Ultimate Strength of Reinforced Concrete Beams as Related to the Plasticity Ratio of Concrete.

No. 346 . . . Highway Slab-Bridges With Curbs: Laboratory Tests and Proposed Design Method.

No. 363 . . . Study of Slab and Beam Highway Bridges. Part I.

No. 369 . . . Studies of Highway Skew Slab-Bridges with Curbs. Part I: Results of Analyses.

No. 375 . . . Studies of Slab and Beam Highway Bridges. Part II.

STATUS OF FEDERAL-AID HIGHWAY PROGRAM

AS OF OCTOBER 31, 1949

(Thousand Dollars)

STATE	UNPROGRAMMED BALANCES	ACTIVE PROGRAM: PROGRAMMED ONLY			ACTIVE PROGRAM: PLANS APPROVED, CONSTRUCTION NOT STARTED			ACTIVE PROGRAM: CONSTRUCTION UNDER WAY			ACTIVE PROGRAM: TOTAL		
		Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles
Alabama	$17,758	$15,443	$7,693	433.0	$3,388	$1,590	67.5	$10,842	$6,295	321.9	$29,673	$15,578	822.4
Arizona	5,821	2,852	2,024	50.5	388	249	9.1	5,901	3,974	60.7	9,141	6,247	120.3
Arkansas	7,870	11,843	6,408	341.1	5,123	2,499	113.3	14,811	7,208	241.2	31,776	16,115	695.6
California	20,695	21,735	8,336	101.6	7,302	3,351	44.8	31,557	15,706	202.1	60,594	27,393	348.5
Colorado	8,306	4,817	2,700	94.6	3,500	1,999	73.6	11,568	6,768	247.9	19,885	11,467	416.1
Connecticut	5,915	9,236	4,306	17.6	622	310	.6	9,128	4,791	19.3	18,986	9,407	37.5
Delaware	4,460	1,607	801	7.2	501	249	.1	2,412	1,736	21.4	4,520	2,786	28.7
Florida	9,965	14,500	7,647	349.5	5,429	2,851	120.7	6,752	2,900	160.9	26,681	13,398	631.1
Georgia	11,624	16,585	8,664	515.6	12,724	5,090	170.6	33,792	17,222	606.9	63,101	30,976	1,293.1
Idaho	7,145	9,138	5,735	325.3	1,379	802	52.0	4,383	2,689	85.7	14,900	9,226	463.0
Illinois	26,332	44,724	24,334	491.8	27,957	12,316	104.3	52,512	26,099	573.6	125,193	62,749	1,169.7
Indiana	20,252	19,542	9,583	101.4	6,445	3,225	19.5	13,724	7,675	76.6	39,711	20,483	197.5
Iowa	12,322	11,333	4,475	459.2	2,720	1,498	83.4	22,208	10,082	738.6	36,261	16,055	1,281.2
Kansas	11,159	10,995	5,361	1,271.2	3,821	1,895	467.7	19,107	9,750	837.6	33,923	17,006	2,576.5
Kentucky	9,909	13,129	6,550	221.8	2,998	1,438	54.1	14,765	7,309	220.1	30,892	15,297	496.0
Louisiana	10,228	22,458	10,627	243.7	9,313	4,867	44.3	18,542	8,432	179.6	50,313	23,926	467.6
Maine	5,747	4,656	2,634	44.5	1,766	887	26.4	5,869	2,930	66.4	12,291	6,451	137.3
Maryland	5,069	4,751	2,291	39.8	1,326	610	5.7	17,953	8,840	60.5	24,030	11,741	106.0
Massachusetts	13,422	31,687	16,523	45.7	7,264	3,445	4.4	18,456	9,543	33.5	57,407	29,511	83.6
Michigan	17,831	13,383	6,671	468.5	5,180	2,569	69.4	40,549	17,242	256.4	59,112	26,482	794.3
Minnesota	12,630	9,547	5,478	577.7	3,754	1,873	59.0	19,252	9,684	400.9	32,553	17,035	1,037.6
Mississippi	15,593	1,190	555	54.4	1,237	610	38.2	14,466	7,393	408.2	16,893	8,558	500.8
Missouri	16,813	28,805	14,509	753.7	10,782	4,767	183.1	25,290	13,174	536.8	64,877	32,450	1,473.6
Montana	11,532	11,562	6,861	432.7	2,604	1,680	98.7	10,674	6,538	224.7	24,840	15,079	756.1
Nebraska	11,176	16,292	8,313	604.6	2,947	1,959	14.9	9,898	5,430	278.6	29,137	15,702	898.1
Nevada	5,360	3,851	3,182	90.4	459	378	11.2	4,018	3,305	92.7	8,328	6,865	194.3
New Hampshire	3,527	5,100	2,535	45.3	53	27		3,542	1,953	18.5	8,695	4,515	63.8
New Jersey	5,094	12,050	4,641	22.0	7,342	3,652	16.5	26,337	12,632	21.1	45,729	20,925	59.6
New Mexico	7,387	8,526	5,473	288.2	1,961	1,254	51.1	3,796	2,511	109.7	14,283	9,238	449.0
New York	59,646	56,057	31,011	243.3	16,378	7,562	19.1	95,340	47,118	269.2	167,775	85,691	531.6
North Carolina	15,418	12,651	6,156	331.8	2,761	1,386	107.9	19,494	9,594	398.7	34,906	17,136	838.4
North Dakota	8,104	6,397	3,397	1,039.4	3,209	1,668	313.7	6,787	3,585	515.5	16,393	8,650	1,868.6
Ohio	23,340	55,948	26,940	344.9	6,637	3,460	39.9	48,584	24,130	186.3	111,169	54,530	571.1
Oklahoma	12,336	25,008	9,413	747.5	7,034	3,694	186.7	11,422	5,701	455.9	43,464	18,808	1,390.1
Oregon	7,068	4,233	2,428	123.0	2,939	1,685	48.1	8,953	4,680	56.3	16,125	8,793	227.4
Pennsylvania	30,817	11,355	5,677	16.2	25,725	13,064	79.6	59,823	29,147	124.6	96,903	47,888	220.4
Rhode Island	3,146	8,689	4,655	19.3	4,311	2,110	4.7	3,404	1,411	6.4	16,404	8,176	30.4
South Carolina	9,521	3,556	1,862	142.2	1,869	1,005	136.9	9,759	4,859	284.8	15,184	7,726	563.9
South Dakota	6,588	9,414	5,590	941.7	1,668	1,081	129.3	8,175	4,926	505.2	19,257	11,597	1,576.2
Tennessee	10,200	14,583	7,163	353.7	3,387	1,617	95.6	20,380	10,358	240.9	38,350	19,138	690.2
Texas	27,799	7,012	3,660	385.0	9,773	4,414	482.4	69,350	30,505	1,286.5	86,135	38,579	2,153.9
Utah	5,148	3,685	2,706	105.2	2,028	1,486	71.8	5,160	3,779	101.0	10,873	7,971	278.0
Vermont	2,627	1,685	843	32.2	1,595	555	6.9	4,034	1,972	41.4	7,314	3,370	80.5
Virginia	15,984	16,158	8,139	473.2	2,443	1,195	78.1	13,317	6,482	178.9	31,918	15,816	730.2
Washington	8,334	13,240	5,666	133.1	2,002	1,013	36.5	14,148	6,382	84.4	29,390	13,061	254.0
West Virginia	4,065	18,211	7,654	203.6	1,774	931	15.0	8,632	4,223	78.7	28,617	12,808	297.3
Wisconsin	18,831	14,227	7,358	327.0	2,330	1,102	86.6	17,822	8,747	407.8	34,379	17,207	821.4
Wyoming	4,573	1,572	1,022	32.8	999	723	18.2	6,948	4,485	236.6	9,519	6,230	287.6
Hawaii	2,615	8,974	3,714	39.9	4,212	1,974	4.8	3,741	1,871	23.5	16,927	7,559	68.2
District of Columbia	2,776	3,370	1,897	.5	700	317	1.0	11,021	5,321	1.5	15,091	7,535	3.0
Puerto Rico	5,714	8,781	3,962	36.9	2,549	1,183	12.0	8,303	2,946	41.0	19,633	8,091	89.9
TOTAL	605,592	686,143	345,823	14,565.0	246,607	121,165	3,979.0	926,701	462,033	12,627.2	1,859,451	929,021	31,171.2

Includes 1951 apportionment.